PREFACE

The **Montana Structures Manual** has been developed to provide uniform structural design practices for Department and consultant personnel preparing contract plans for Department projects. The bridge designer should attempt to meet all criteria presented in the **Manual**. However, the **Manual** should not be considered a "standard" which must be met regardless of impacts.

The **Manual** presents much of the information normally required for a structural project; however, it is impossible to address every situation which the bridge designer will encounter. Therefore, designers must exercise good judgment on individual projects and, frequently, they must be innovative in their approach to structural design. This may require, for example, additional research into the structural design literature.

The **Montana Structures Manual** was developed by the MDT Bridge Design Section with assistance from the engineering consulting firm of Roy Jorgensen Associates, Inc. and its subcontractor SRD Engineering. The **Manual** Review Committee consisted of:

William Fullerton (Project Coordinator)	Montana Department of Transportation
Kent Barnes	Montana Department of Transportation
Dave Johnson	Montana Department of Transportation
Kevin McCray	Montana Department of Transportation
Nigel Mends	Montana Department of Transportation
Bryan Miller	Montana Department of Transportation
Bob Modrow	Montana Department of Transportation
Devin Roberts	Montana Department of Transportation
Merrill Rutherford	Montana Department of Transportation
Ken Shearin	Roy Jorgensen Associates, Inc
Craig Birkholz	Roy Jorgensen Associates, Inc.
Randall Burks	Roy Jorgensen Associates, Inc.
Dennis Mertz	University of Delaware

Title: Structures Manual / Montana Dept. of Transportation

Volume II - Structural Design

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Chapter Eleven

GENERAL

Volume II "Structural Design" of the **MDT Structures Manual** presents the Department's criteria for the structural design of bridges and other structures. Chapter Eleven presents general information which applies to all of Volume II.

11.1 BASIC APPROACH

The following describes the basic approach used to develop Volume II of the **Manual**:

1. Application. The **MDT Structures Manual** is intended to be an application-oriented product.
2. Theory. The **Manual** is not intended to be a structural design theory resource nor a research document. The **Manual** will only provide background information as absolutely necessary so that the user will understand the basis for the Department's structural design criteria and application.
3. Example Problems. Where beneficial to explain the intended application, the **Manual** will provide example problems to demonstrate the proper procedure for structural design.
4. Details. Where beneficial, the **Manual** will provide structural design details (i.e., figures and tables) for the various structural components.
5. Coordination with **LRFD Bridge Design Specifications**. A crucial element to **Manual** development is the **Manual's** coordination with the LRFD Specifications. The **MDT Structures Manual** is basically a Supplement to the LRFD Specifications which:

   a. in general, does not duplicate, unless absolutely necessary for clarity, information in the AASHTO Specifications;

   b. elaborates on specific articles of the Specifications;

   c. presents interpretative information, where required;

   d. modifies sections from the Specifications where the Department has adopted a different practice;

   e. where the AASHTO Specifications presents more than one option, indicates the Department's preference; and

   f. indicates structural design elements presented in the AASHTO Specifications but which are not typically used in Montana.

6. Acknowledgement of 16^{th} Edition of **Standard Specifications for Highway Bridges**. This **Manual** has been prepared with the expectation that the **LRFD Bridge Design Specifications** will replace the 16^{th} Edition of the AASHTO **Standard Specifications for Highway Bridges**. The reality is that the implementation of the LRFD Specifications is a transitional process. Much design is still being prepared using the 16^{th} Edition and will be for some time. If a bridge design is being prepared using the 16^{th} Edition of the Standard Specifications, the information and guidance provided in this **Manual** needs to be considered in conjunction with the requirements of that publication and used as appropriate.

11.2 STRUCTURAL DESIGN LITERATURE

Section 11.2 discusses the major national publications available in the structural design literature. It provides 1) a brief discussion on each publication, and 2) the status and application of the publication by the Department. Section 11.2 is not all inclusive of the structural design literature; however, it does represent a hierarchy of importance. In all cases, the designer must ensure that he/she is using the latest edition of the publication, including all interim revisions to date.

11.2.1 LRFD Bridge Design Specifications

11.2.1.1 Description

The AASHTO **Load and Resistance Factor Design (LRFD) Bridge Design Specifications** are intended to serve as the national standard or guide for use by bridge engineers or for the development of a transportation agency's own structural specifications. The Specifications establish minimum requirements, consistent with current nationwide practices, which apply to common highway bridges and other structures such as retaining walls and culverts; large-span structures may require design provisions in addition to those presented in the LRFD Specifications. Because of the continually changing nature of structural design, interim revisions are issued and, periodically, AASHTO publishes a completely updated edition, historically at four-year intervals.

The LRFD Specifications take a fundamentally different approach to design theory than the AASHTO **Standard Specifications for Highway Bridges**. The information in the LRFD Specifications supersedes, partially or completely, the following AASHTO structural design publications:

1. **Standard Specifications for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections**,
2. **Guide Specifications for Strength Design of Truss Bridges**,
3. **Standard Specifications for Seismic Design of Highway Bridges**,
4. **Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members**,
5. **Guide Specifications — Thermal Effects in Concrete Bridge Superstructures**,
6. **Guide Specifications for Fatigue Design of Steel Bridges**,
7. **Guide Specifications for Bridge Railings**,
8. **Guide Specifications for Design and Construction of Segmental Concrete Bridges**, and
9. **Guide Specification and Commentary for Vessel Collision Design of Highway Bridges**.

The LRFD Specifications present a load and resistance factor methodology for the structural design of bridges, which replace the load factor and allowable stress methodologies of the previous AASHTO Standard Specifications. The LRFD Specifications apply live load factors which are lower than the traditional AASHTO load factors but balance this reduction with an increase in vehicular live load which more accurately models actual loads on our nation's highways. Basically, the LRFD methodology requires that bridge components be designed to satisfy four sets of limit states: strength, service, fatigue and extreme-event limit states. Through the use of statistical analyses, the provisions of the LRFD Specifications reflect a uniform safety index for all structural elements, components and systems.

A few significant features of the LRFD Specifications are:

1. The Specifications are supplemented with a comprehensive commentary placed immediately adjacent to the Specifications provisions in a parallel column.
2. The vehicle live load is designated HL-93. This live load model retains a truck

configuration similar to the HS-20 design truck and a tandem slightly heavier than the traditional military loading, but it has been modified to include simultaneously applied lane loading over full or partial span lengths to produce extreme force effects.

3. Alternative load factors have been introduced for permanent loads that must be used in combination with factored transient loads to produce extreme force effects.

4. Fatigue loading consists of a single truck with axle weights and spacings that are the same as an HS-20 truck with a constant 9-m spacing between the 142-kN axles which can be located anywhere on the bridge deck to produce the maximum stress range.

5. In addition to regular load combinations, two design trucks are used for negative moments and internal pier reactions in combination with the lane load, the distance between the rear and front axles of the trucks cannot be less than 15 m, and the combined force effect is reduced by 10%.

6. The Specifications include an empirical design for concrete bridge deck slabs, which allows for reduced deck reinforcement.

7. The Specifications allow for relatively easy and more precise estimates of live-load distribution by tabulated equations.

8. The Specifications allow the optional use of deflection criteria. See Section 11.3.4 for Departmental guidance.

9. The Specifications allow for the more frequent use of compact steel sections.

10. The method of shear design in concrete has been revised; compression field theory and strut-and-tie models are used.

11. The Specifications recognize the detrimental effect of salt-laden water seeping through deck joints and promote the notion of reducing the number of such joints to an absolute minimum.

11.2.1.2 Department Application

The Montana Department of Transportation has adopted the use of the AASHTO **LRFD Bridge Design Specifications** as the preferred document for the structural design of highway bridges in Montana. Volume II of the **Montana Structures Manual** presents the Department's specific application of the LRFD Specifications to structural design, which modify, replace, clarify or delete information from the AASHTO LRFD Specifications for MDT's application.

The AASHTO **Standard Specifications for Highway Bridges** are still valid and may be used with the approval of the Bridge Area Engineer. This **Manual** contains significant information on Department design policies and procedures and must be referenced for designs prepared based on the **Standard Specifications for Highway Bridges**.

Where conflicts are observed in the structural design literature, the following hierarchy of priority shall be used to determine the appropriate application:

1. **Montana Structures Manual,**

2. **LRFD Bridge Design Specifications** or **Standard Specifications for Highway Bridges,** and

3. all other publications.

11.2.2 ANSI/AASHTO/AWS Bridge Welding Code (1996 Edition) D1.5

11.2.2.1 Description

The **Bridge Welding Code** presents current criteria for the welding of structural steel in bridges. The Code superseded the 1981 AASHTO **Standard Specifications for Welding of**

Structural Steel Highway Bridges and the 1980 **Structural Welding Code, AWS D1.1.**

11.2.2.2 Department Application

The Department has adopted the use of the 2002 **Bridge Welding Code D1.5** for the design and construction of structural steel highway bridges. However, the D1.5 Code does not cover welding on reinforcing steel or welding on existing structures. For these items, refer to the current edition of ANSI/AWS D1.1, ANSI/AWS D1.4 and ANSI/AWS D1.1, respectively.

11.2.3 Standard Specifications for Seismic Design of Highway Bridges (1994 Edition)

11.2.3.1 Description

The AASHTO Seismic Specifications present design criteria for the seismic design of highway bridges to, within reason, limit significant structural damage or structural failure of a highway bridge during an earthquake. The AASHTO Seismic Specifications are applicable throughout the United States, with varying levels of risk assigned to different areas of the nation based upon seismicity. It is based upon the observed performance of bridges during earthquakes and upon research which has been conducted worldwide.

As noted in Section 11.2.1, the AASHTO LRFD Specifications have incorporated and supersede the Seismic Specifications. However, the Seismic Specifications contain useful information discussing background and methods of analysis, and they provides worked examples which are by nature not incorporated into the LRFD Specifications.

In 1999, AASHTO published the **Guide Specifications for Seismic Isolation Design** (Second Edition), which was supplemental to the **Standard Specifications for Seismic Design of Highway Bridges**. The LRFD Specifications do not specifically address seismic isolators; therefore, these Specifications may be used in conjunction with the LRFD Specifications.

11.2.3.2 Department Application

The AASHTO **Standard Specifications for Seismic Design of Highway Bridges** may be used by the designer for informational purposes. The **Guide Specifications for Seismic Isolation Design** should be used, where applicable, in conjunction with the LRFD Specifications.

11.2.4 Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members (1996 Edition)

11.2.4.1 Description

The AASHTO Guide Specifications provide recommended requirements for identifying, fabricating, welding and testing of fracture critical, non-redundant steel bridge members whose failure would be expected to cause a bridge to collapse. The AASHTO Guide Specifications include specifications on welding requirements which are in addition to those in the ANSI/AASHTO/AWS **Bridge Welding Code**. The Guide also discusses the need for proper identification of fracture critical members on plans.

As noted in Section 11.2.1, the AASHTO LRFD Specifications have incorporated and supersede the Guide Specifications. See Article 6.6.2. However, the Guide Specifications contain useful information addressing background, example problems, etc., which are not included in the LRFD Specifications.

11.2.4.2 Department Application

The AASHTO Guide Specifications may be used by the designer for informational purposes.

11.2.5 Guide Specifications for Horizontally Curved Highway Bridges (2002 Edition)

11.2.5.1 Description

The AASHTO Guide Specifications present specifications and methodologies for the design of steel beams and steel box girder bridges which are on a horizontal curve. The Guide is applicable to simple and continuous spans, composite or non-composite structures of moderate length employing either rolled or fabricated sections. The design methodology is based on both working stress and load factor principles and, therefore, is not compatible with the LRFD Specifications.

11.2.5.2 Department Application

The Department has adopted the AASHTO Guide Specifications as standard practice; therefore, they shall be used for the design of horizontally curved steel members.

11.2.6 Guide Specifications For Bridge Railings (1989 Edition)

11.2.6.1 Description

The AASHTO Guide Specifications contain three bridge railing performance levels and associated crash tests and performance requirements in addition to guidance for determining the appropriate railing performance level for a given bridge site.

As noted in Section 11.2.1, the AASHTO LRFD Specifications have incorporated and supersede the Guide Specifications. However, the Guide Specifications contain useful information addressing background, example problems, etc., which are not included in the LRFD Specifications.

11.2.6.2 Department Application

The AASHTO Guide Specifications may be used by the designer for informational purposes.

11.2.7 Guide Specifications for Structural Design of Sound Barriers (1989 Edition)

11.2.7.1 Description

The AASHTO Guide Specifications provide criteria for the structural design of sound barriers to promote the uniform preparation of plans and specifications. The AASHTO Guide Specifications allow the design of masonry sound barriers in addition to concrete, wood, steel, synthetics and composites, and aluminum.

11.2.7.2 Department Application

Use the AASHTO Guide Specifications for all sound barrier designs.

11.2.8 Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals (1994 Edition with Interim Revisions)

11.2.8.1 Description

The AASHTO Standard Specifications present structural design criteria for the supports of various roadside appurtenances. The publication presents specific criteria and methodologies for evaluating dead load, live load, ice load and wind load. The AASHTO Standard Specifications also include criteria for several types of materials used for structural supports such as steel, aluminum, concrete and wood.

11.2.8.2 Department Application

The Department has adopted the use of the AASHTO **Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals**. The MDT Traffic Engineering Section is primarily responsible for these supports, and the Department has developed standard designs which will apply in most cases.

11.2.9 LRFD Manual of Steel Construction

11.2.9.1 Description

The **LRFD Manual of Steel Construction**, published by the American Institute of Steel Construction (AISC), provides dimensions, properties and general design guidance for structural steel for various applications. The Manual contains AISC criteria for steel buildings. However, the properties of the rolled structural shapes are useful for designing bridge structures.

11.2.9.2 Department Application

The designer may use the **AISC LRFD Manual of Steel Construction** for informational purposes.

11.2.10 Timber Construction Manual

11.2.10.1 Description

The **Timber Construction Manual**, published by the American Institute of Timber Construction (AITC), provides comprehensive criteria for the design of timber structures, including bridges. The Manual contains information for both sawn and laminated timber.

11.2.10.2 Department Application

The designer should use the AITC **Timber Construction Manual** to supplement the AASHTO publications on the design of timber bridges.

11.2.11 Uniform Building Code

11.2.11.1 Description

The **Uniform Building Code** (UBC), published by the International Conference of Building Officials (ICBO), provides criteria for the design of buildings throughout the United States and abroad. They are intended to be used directly by an agency or to be used in the development of an agency's own building codes. Contact:

> Building Codes Bureau
> MT Department of Labor and Industry
> PO Box 200517
> Helena, MT 59620-0517
> (406) 841-2056

to determine which version of the UBC to use as a reference.

11.2.11.2 Department Application

Buildings for which the Department is responsible for their design (e.g., at rest areas) shall be designed based on the **Uniform Building Code.**

11.2.12 AREMA Manual

11.2.12.1 Description

The **AREMA Manual**, published by the American Railroad Engineering and Maintenance-of-Way Association (AREMA), provides detailed structural specifications for the design of railroad bridges. The AREMA specifications have approximately the same status for railroad bridges as the LRFD Specifications have for highway bridges; i.e., the structural design of railroad bridges shall meet the AREMA requirements.

11.2.12.2 Department Application

In some cases, MDT is responsible for the design of railroad bridges over highways. The specifications of the **AREMA Manual** must be met, except as modified by railroad companies operating in Montana.

11.2.13 Other Structural Design Publications

The structural design literature contains many other publications which may, on a case-by-case basis, be useful. These may be used at the discretion of the designer. The following briefly

describes several other structural design publications:

1. **Prestressed Concrete Institute (PCI) Design Handbook**. This publication includes information on the analysis and design of precast and/or prestressed concrete products in addition to a discussion on handling, connections and tolerances for prestressed products. It contains general design information, specifications and standard practices.

2. **Post-Tensioning Institute (PTI) Post-Tensioning Manual**. This publication discusses the application of post-tensioning to many types of concrete structures, including concrete bridges. The publication also discusses types of post-tensioning systems, specifications, the analysis and design of post-tensioned structures and their construction.

3. **Concrete Reinforcing Steel Institute (CRSI) Handbook**. This publication meets the ACI Building Code Requirements for Reinforced Concrete. Among other information, it provides values for both design axial load strength and design moment strength for tied columns with square, rectangular or round cross sections, and it provides pile cap designs.

4. **National Steel Bridge Alliance (NSBA) Highway Structures Design Handbook.** This document addresses many aspects of structural steel materials, fabrication, economy and design. Recently updated with LRFD examples in both US customary units and SI units, the general computational procedure is helpful to designers using the **LRFD Bridge Design Specifications.**

5. **United States Department of Agriculture (USDA) Forest Service Timber Bridge Manual**. This is a comprehensive document covering all aspects of traditional timber bridge construction plus the latest developments in laminated deck systems using adhesives or prestressing forces.

6. **Western Lumber Grading Rules**. This publication contains information on how lumber and timbers are graded, and it has stress tables for various species, grades and sizes.

7. **West Coast Lumber Grading Rules**. This publication contains information on how lumber and timbers are graded, and it has stress tables for various species, grades and sizes.

8. **American Concrete Institute (ACI) — Analysis and Design of Reinforced Concrete Bridge Structures.** This publication contains information on various concrete bridge types, loads, load factors, service and ultimate load design, prestressed concrete, substructure and superstructure elements, precast concrete, reinforcing details and metric conversion.

9. **CRSI Manual of Standard Practice**. This publication explains generally accepted industry practices for estimating, detailing, fabricating and placing reinforcing bars and bar supports. MDT requires that reinforcing steel shall be detailed as shown in the CRSI Manual as modified by MDT practices.

10. **PTI — Post-Tensioned Box Girder Bridges.** This publication contains information on economics, design parameters, analysis and detailing, installation, prestressing steel specifications, post-tensioning tendons, systems and sources.

11. **United States Navy — Design Manual for Soil Mechanics, Foundations and Earth Structures.** This is a comprehensive document covering embankments, exploration and sampling, spread footings, deep foundations, pressure distributions, buried substructures, special problems, seepage and drainage analysis, settlement analysis, soil classifications, stabilization, field tests and measurements, retaining walls, etc. Its use is strongly recommended. Note that the loading sections of the Manual are superseded by the LRFD Specifications.

12. **Seismic Design and Retrofit Manual for Highway Bridges**, Report No. FHWA-IP-87-6. This Manual contains information concerning basic seismology, bridge dynamics, design concepts, loads, forces and displacements in addition to design examples, retrofitting and comparative analyses.

13. **Seismic Design of Bridges**, Report No. FHWA-SA-97-007. This is a series of seven worked examples on seismic design. Each volume contains one worked example.

11.3 GENERAL STRUCTURAL DESIGN CRITERIA

11.3.1 General

Reference: None

The girders should be designed, where practical, so that exterior and interior girders will be similar to allow for the possibility of future widening. This also reduces fabrication costs and the probability for misplacement.

11.3.2 Continuity

Reference: LRFD Article 2.5.2.4

Continuity within bridge structures is a very desirable objective because, among many other advantages, the number of deck joints decreases with deck continuity.

Article 2.5.2.4 contains strong wording on using bridge approach slabs. These are not normally used on MDT projects. Approach slabs are required where Portland cement concrete pavement (PCCP) is used or for special considerations.

11.3.3 Composite Action

Reference: LRFD Article 9.4.1

The Department mandates the use of composite action between the superstructure and the bridge deck on new construction. On a project-by-project basis, investigate the potential benefits or consequences of composite action when rehabilitating existing bridges.

11.3.4 Deflection Criteria

Reference: LRFD Article 2.5.2.6.2

The LRFD Specifications make the traditional live-load deflection criteria optional for both bridges with and without sidewalks because static live-load deflection is not a good measure of dynamic excitation. Nonetheless, in the absence of a better criterion and because of concerns on deck life, the MDT believes that it is appropriate to limit deflections.

11.3.4.1 Structures With Sidewalks

Stringers or girders having simple or continuous spans shall be designed so that the deflection due to truck live load plus dynamic allowance shall not exceed 1/1000 of the span. The deflection of cantilever arms due to live load plus dynamic allowance shall be limited to 1/375 of the cantilever arm.

11.3.4.2 Structures Without Sidewalks

The deflection allowance shall not exceed 1/800 of the span length.

11.3.5 Semi-Integral Abutments

Reference: None

Semi-integral abutments are the typically preferred abutment type in Montana, because they offer many of the advantages of integral abutments while offering additional advantages in construction and maintenance. In severe seismic applications, integral abutments are preferred. Chapter 19 discusses the advantages of each abutment type and provides MDT practices.

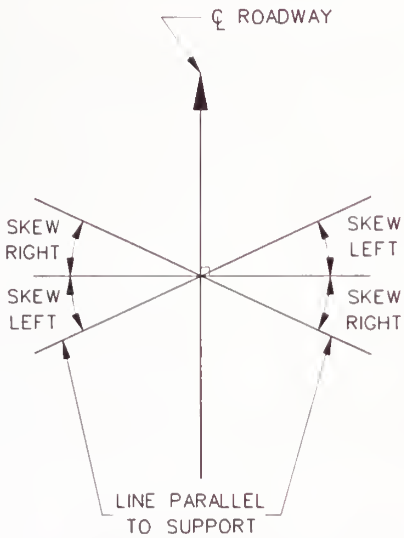
11.3.6 Skew

The following applies to skews on bridges:

1. Definition. Skew angle is defined as the angle between a line normal to the highway centerline (or a tangent thereto) and a line parallel to the support (wall, abutment, pier, etc.). The angle is designated left or right, in relation to the normal to the highway centerline. See Figure 11.3A.

2. Snowplows. If possible, avoid matching the angle of a snowplow. The critical angle to avoid is 35° - 37° right.

3. Department Limits. The maximum skew angle on a bridge without approval is 35°. The Bridge Area Engineer must approve the use of greater skew angles.



SKEW DEFINITION
Figure 11.3A

11.4 EXCEPTIONS

Section 11.4 discusses the Department's procedures for identifying, justifying and processing exceptions to the structural design criteria in the **MDT Structures Manual**.

11.4.1 Department Intent

The general intent of the Montana Department of Transportation is that all design criteria in this **Manual** and the **LRFD Bridge Design Specifications** or the **Standard Specifications for Highway Bridges** shall be met. This is intended to ensure that the Department will provide a highway system which meets the transportation needs of the State and provides a reasonable level of safety, durability, comfort and convenience for the traveling public. However, recognizing that this may not always be practical, the Department has established a process to evaluate and approve exceptions to its structural design criteria.

11.4.2 Procedures

Where the bridge designer proposes a design element which does not meet the requirements of the **MDT Structures Manual**, regardless of whether or not it satisfies the provisions of the **LRFD Bridge Design Specifications** or the **Standard Specifications for Highway Bridges**, the following procedure will apply:

1. Documentation. The bridge designer will document any proposed exceptions from the Department's structural design criteria in the Scope of Work Report, if known at this stage of project development. The designer will present the justification for the exception, which may include:

    a. site constraints,
    b. construction costs,
    c. environmental impacts, and/or
    d. right-of-way impacts.

2. Approval. All proposed exceptions must be approved by the Bridge Design Engineer or Bridge Engineer.

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Chapter Twelve
STATE PLANE COORDINATE SYSTEM

Chapter Twelve discusses the application of the Montana State Plane Coordinate System in highway and bridge projects and presents guidelines for employing this system during design and plan preparation. Chapter Twelve provides the designer with:

1. a discussion of the purpose of survey datums and of the vertical and horizontal control datums that are typically used by the Department;

2. information on location and survey methods that illustrates the need and benefit of employing a state plane coordinate system in highway projects;

3. a discussion of the fundamental elements of the state plane coordinate system that is adopted by the State of Montana and employed by the Department in its program of projects;

4. a discussion explaining the relationship between the ground surface, the surface of the Geodetic Reference System 1980 (i.e., GRS 80) ellipsoid and that of the state plane grid;

5. an explanation of why a distance between points on the ground differs in magnitude from the distance between the same points as they are projected onto the state plane grid; and

6. guidelines for working within the Montana State Plane Coordinate System during bridge design and plan preparation.

For additional information on survey datums, coordinate systems and related mathematical computations, the designer is referred to the Department's **Surveying Manual**.

12.1 SURVEY DATUM CONSIDERATIONS

This Section provides the designer with a discussion of the purpose of survey datums and of the vertical and horizontal control datums that are typically used by the Department.

12.1.1 Purpose of Survey Datums

A survey datum is a numeric benchmark that serves as a control reference from which other measurements can be made during design, surveying and construction. A survey control datum may be either:

1. assumed,
2. given,
3. measured, or
4. otherwise determined.

Highway and bridge projects typically require design and survey measurements to reference both a vertical and horizontal datum. Their purpose is to provide the necessary control to accurately measure and survey project elements (e.g., horizontal and vertical curves, grades, bridge decks). By referencing an accurate and well-established datum, costly mistakes can be avoided and relationships between and connections to adjoining projects can be readily established.

12.1.2 Vertical Control Datum

A vertical datum is used to accurately establish and control the elevations of and between elements in the project. Where practical, the Department references the North American Vertical Datum 1988 (i.e., NAVD 1988) for vertical control in its program of projects. When used, this vertical datum is employed consistently in both design and construction.

The datum reference will appear in both the field notes and the plan sheets. The following are cases where other vertical datum may be utilized:

1. Urban Projects. Some urban projects may require that vertical control be tied to a local, city or county datum that does not reference NAVD 1988; however, if a local datum is used, the field notes and plans will clearly state the origin of the vertical datum.

2. Small, Remote Projects. As determined on a case-by-case basis, it may be necessary to utilize an assumed datum for vertical control on small, remote projects where the expense of tying the project to NAVD 1988 is not justified. The project manager will verify the acceptability of using another datum type.

12.1.3 Horizontal Control Datum

A horizontal datum is used to accurately establish and control ground locations, lengths and angles of and between project elements during design, surveying and construction. Horizontal control is typically employed on a coordinate-based grid system, either assumed (e.g., local coordinate) or previously established (e.g., state plane). Such control is used in highway engineering to:

1. define positions of survey stations,
2. locate points-of-intersection,
3. define lines and curves, and
4. compute lengths and azimuths.

Where practical, the Department references the Montana State Plane Coordinate System (see 70-22-201 **Montana Code Annotated**). This state plane coordinate system employs the North American Datum 1983 (i.e., NAD-83) Single Zone Coordinate System. The NAD-83 system is based on computations-of-position on a precise geodetic reference ellipsoid that approximates the shape of the earth's surface at sea level. This earth-centered ellipsoid is known as the Geodetic Reference System of 1980 (i.e., GRS 80) and is established and maintained by the National Geodetic Survey using very precise, and expensive, geodetic-surveying methods.

When used, the Montana State Plane Coordinate System is employed consistently in both design and construction. The project manager will verify the acceptability of using another datum type. The datum reference and information relative to the application of the state plane coordinate system will appear in the field notes and plan sheets.

12.2 LOCATION AND SURVEY METHODS

This Section presents information on location and survey methods that illustrate the need and benefit of using a state plane coordinate system in highway projects.

12.2.1 Limitations of Plane Surveying

A conventional survey that employs plane-surveying methods and uses a horizontal control datum that is based on a local coordinate system is both cost-effective and relatively simple to implement. However, such a methods's limitations are substantial when employed in highway projects as documented in the following.

12.2.1.1 Effects of the Earth's Curvature

Plane-coordinate systems do not take into account the affects of the earth's curvature. This is typically not a major concern for surveys of limited scope; however, as horizontal control is extended from the origin of the coordinate grid, the effects of the earth's curvature become more apparent and critical.

Conventional plane-surveying methods are based on the assumption that all distances and directions are projected onto a horizontal plane that is tangent to the earth's surface at one point within the survey area. Because of the earth's curvature, as the survey departs from the point-of-tangency, there will be an increasing difference between the ground distance and the distance that results from projecting the defining points onto the coordinate grid of the horizontal plane. This deviation becomes intolerable for surveys that traverse a great distance (e.g., highway projects). When this deviation becomes intolerable, then the limits of conventional plane-surveying methods have been reached.

12.2.1.2 Relationships of Independent Surveys

Attempting to establish a relationship between multiple plane surveys, each having been conducted independently on a different horizontal datum, is both difficult and ineffective. This limitation becomes critical when attempting to connect adjoining highway projects.

If made independently, plane surveys will reference different, non-coinciding coordinate-grid systems. Furthermore, if the north-south oriented axis of each survey's coordinate system is assumed to be parallel to the true meridian (i.e., longitude) at one station within the survey, then their axes will not even be parallel due to the convergence of the meridians.

12.2.2 Positioning By Latitude and Longitude

Latitude and longitude are geographic positions that are earth-centered angular measurements. Such measurements are expressed in units of degrees, minutes and decimal seconds. Latitude is the north or south location of a point referenced from the equator. Longitude is the east or west location of a point referenced from the meridian that passes through Greenwich, England. For example, Beartooth Pass, Montana is located at approximately 45° 00′ 17″ North Latitude and 109° 24′ 36″ West Longitude. While positioning by latitude and longitude is useful for navigation, it is not practical nor convenient to employ as a method-of-location in highway engineering work.

12.2.3 Benefits of Geodetic Surveying

In the methods of geodetic surveying, all distances are reduced to a common reference surface that conforms closely to sea level. Coordinates of points are computed with reference to parallels of latitude and meridians of longitude by using angles computed near the center of the earth rather than distances.

Geodetic surveying is employed so that precise surveys may be extended over great distances in any direction without suffering the limitations of plane-surveying methods; however, geodetic-surveying methods are:

1. more complex and more expensive,
2. involve more difficult computations, and
3. require specialized personnel in execution.

The National Geodetic Survey has established and maintains horizontal control monuments in the form of triangulation, traverse and intersection stations. These stations have been located by the methods of geodetic surveying. The control points in this network are continually being updated and bear a definite relationship, one to another, being referred to one common reference surface.

12.2.4 State Plane Coordinate Systems

State plane coordinate systems are made possible by virtue of the highly precise geodetic-surveying methods used in establishing the initial framework of the horizontal control that defines these systems. This horizontal control can be readily incorporated into conventional surveys for the purposes of:

1. coordinating,
2. checking,
3. establishing, and
4. reestablishing survey points.

Employing a state plane coordinate system allows the relatively simple methods of plane surveying to be used over great distances in any direction while accounting for the affects of the earth's curvature and maintaining a precision approaching that of geodetic surveying. For the surveys that are tied to a state plane coordinate system, each will be related to one another. This is the most singular advantage of a state plane coordinate system. Other advantages of using a state plane coordinate system are as follows:

1. Greater Distances. The use of a state plane coordinate system permits conventional surveys to be carried over Statewide distances by using plane-surveying methods with results that approach that obtained by geodetic-surveying methods.

2. Project Conformance. Plans which have been controlled by coordinated points on a state plane will always conform when joined, no matter how unrelated the projects were that necessitated the plans.

3. Consistent Basis of Bearing. By tying a project to a state plane coordinate system, especially where multiple surveys are involved, a consistent basis of bearing may be established.

4. GIS Project Database. By employing a state plane coordinate system, a Geographic Information System (i.e., GIS) database of projects can be established and maintained for an entire state whether the projects are conducted by a state or a local government agency.

5. Consistent Use of GPS. The deployment of the satellite-based Global Positioning System (i.e., GPS) greatly simplifies obtaining known positions for conversion to State plane coordinates. GPS allow the expanded use of State plane coordinates in projects.

6. Increase in Accuracy. A traverse of relatively low accuracy that is run between a pair of control points is actually raised in accuracy after an adjustment between the control points is made.

7. Reduction in Error. The use of well-established control points in a traverse eliminates serious errors in measuring both distances and angles.

8. Fewer Control Surveys. The use of a common reference system reduces or eliminates the costly duplication of control surveys possibly required by multiple projects in the same area.

9. Reestablishment of Lost Points. A point whose coordinates have been determined

can, if lost, always be replaced with the degree of precision in which it was originally established

10. Aerial Control/Mapping. Photogrammetric mapping can be conducted at much less expense when all control points in the area to be mapped are on the same system. State plane coordinates are shown on many federal maps, particulary United States Geological Survey Quadrangle Topographic Maps.

12.3 THE MONTANA STATE PLANE COORDINATE SYSTEM

This Section discusses the fundamental elements of the state plane coordinate system used by the State of Montana and the Department.

12.3.1 Montana State Statute

The Montana State Legislature has enacted laws establishing the legal statute of the Montana State Plane Coordinate System. The designer is referred to §70-22-201 of the **Montana Code Annotated** for a definition of the Montana State Plane Coordinate System, its use and limitations.

12.3.2 Geodetic Reference Datum

The geodetic reference system used for the Montana State Plane Coordinate System is the Geodetic Reference System of 1980 (i.e., GRS 1980). This reference system is maintained by the National Geodetic Survey using the very precise methods of geodetic surveying. GRS 80 is an earth-centered ellipsoid that approximates the shape of the earth's surface at sea level. The computations-of-positions on this ellipsoid are known as the North American Datum of 1983 (i.e., NAD-83). The State of Montana has elected to develop its state plane coordinate system based on the NAD-83 Single Zone Coordinate System.

The National Geodetic Survey publishes NAD-83 coordinates in the metric system-of-units (i.e., meters). The Department publishes NAD-83 coordinates in either meters or positions of latitude and longitude. The Montana State Plane Coordinate System specifies that the conversion factor that should be used to convert between the English and metric systems is the international conversion factor of 1 ft = 0.3048 m.

12.3.3 Lambert Conformal Projection

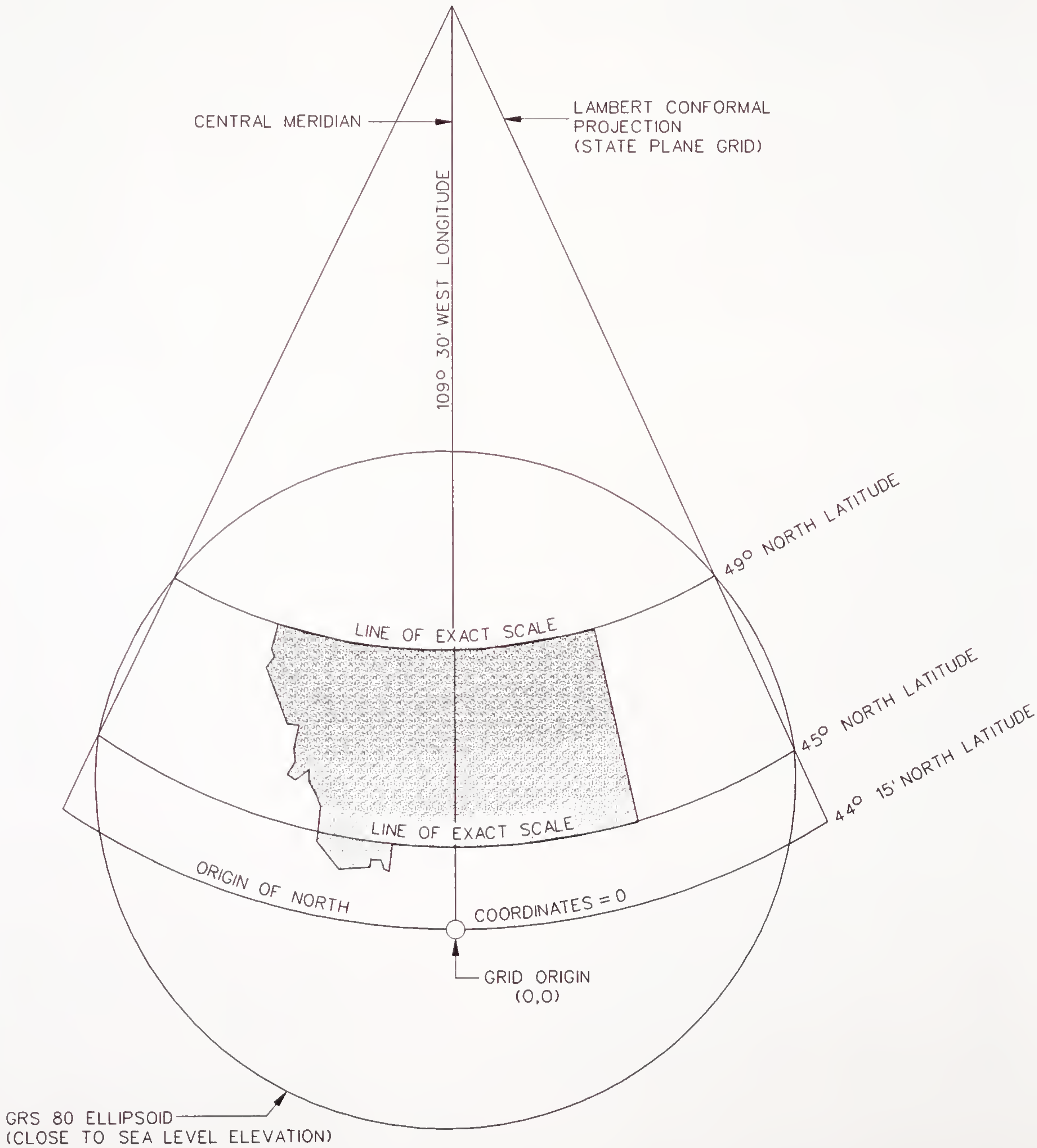
The Montana State Plane Coordinate System is based on the NAD-83 Single Zone Coordinate System and is developed through use of the Lambert conformal projection as represented diagrammatically in Figure 12.3A. For Montana, the NAD-83 point-of-origin is defined as 44° 15′ North Latitude and 109° 30′ West Longitude (i.e., the State's central meridian). The projection employs a conical surface with its axis coinciding with that of the earth's rotation. The GRS 80 ellipsoid (i.e., that surface approximating the earth's surface at sea level) is intersected by the cone along two parallels of latitude known as the lines of exact scale (i.e., 49° and 45° North Latitude). These lines are approximately equidistant from a parallel lying in the center of the State. The state plane grid is actually developed by mathematically projecting positions of latitude and longitude from the GRS 80 ellipsoid onto the conical surface (i.e., the state plane). Montana uses the Lambert conformal projection as distortions occur over the State's relatively short north-south direction.

Along the lines of exact scale, distances on the state plane grid are the same as corresponding distances on the GRS 80 ellipsoid. Between lines of exact scale, a distance on the state plane grid is smaller than the corresponding distance on the GRS 80 ellipsoid. Outside the lines of exact scale, a projected distance is larger. The discrepancy between corresponding distances depends on the position of the line being considered with respect to the lines of exact scale as follows:

- The scale of a line running in a north-south direction varies from point-to-point.
- A due east-west line has a constant scale along its length, whether it be larger than, equal to or less than that scale corresponding to the GRS 80 ellipsoid.

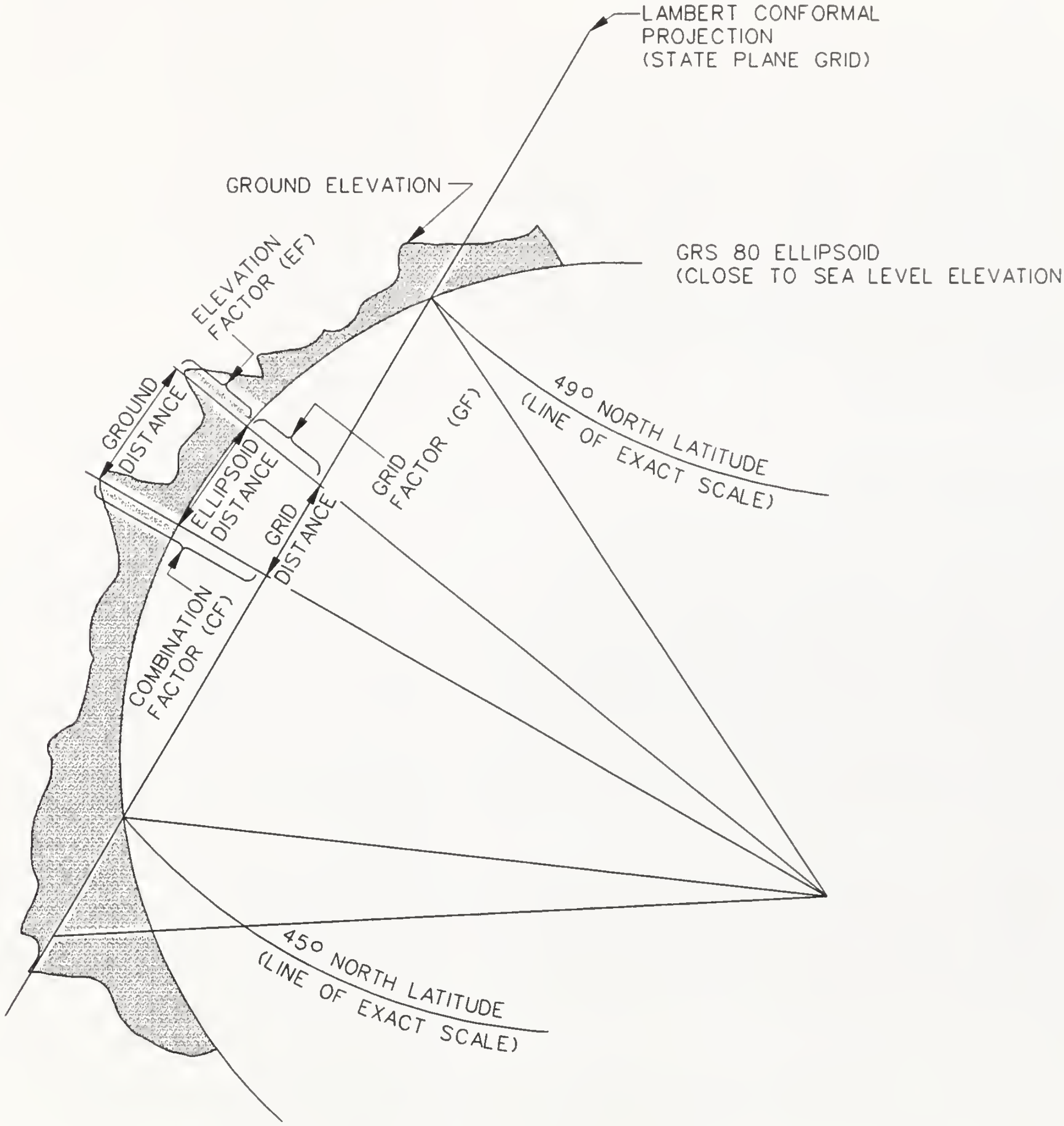
12.3.4 Projections on the State Plane Grid

Figure 12.3B presents a diagram that illustrates the relationship of a distance as it appears during projection on:



THE LAMBERT CONFORMAL PROJECTION OF THE MONTANA STATE PLANE GRID

Figure 12.3A



RELATIONSHIP BETWEEN THE GROUND SURFACE, THE GRS 80 ELLIPSOID AND THE STATE PLANE GRID

Figure 12.3B

- the ground surface;
- the surface of the Geodetic Reference System 1980 (i.e., GRS 80) ellipsoid; and
- the surface of the state plane grid.

As illustrated by Figure 12.3B, a distance between points on the ground will differ in magnitude from the distance between the same points as they are projected onto the state plane grid. This fact necessitates the use of scaling factors to convert a corresponding distance as it is projected through the three surfaces. The following Section defines the scaling factors that are typically employed (i.e., grid, elevation and combination factors).

12.3.4.1 Grid Factor

The grid factor (GF) is a dimensionless scale, or multiplication, factor that is used to convert a distance between points on the GRS 80 ellipsoid (i.e., at approximately sea level) to an equivalent distance between the points as they are projected onto the state plane grid (i.e., on the Lambert conformal projection). The following defines the ranges in magnitude of the grid factor that can be expected in Montana:

1. GF < 1.0. At locations between lines of exact scale (i.e., between 49° North and 45° North Latitudes), the grid factor is less than 1.0. This region encompasses the majority of locations in the State.
2. GF = 1.0. The grid factor is equal to 1.0 at locations along lines of exact scale (i.e., along either 49° North or 45° North Latitudes).
3. GF > 1.0. At locations below 45° North Latitude (i.e., the southwest tip of the State), the grid factor is greater than 1.0.

12.3.4.2 Elevation Factor

The elevation factor (EF) is a dimensionless scale, or multiplication, factor that is used to convert a distance between points on the ground to an equivalent distance between the points as they are projected onto the GRS 80 ellipsoid (i.e., at approximately sea level). The following defines the ranges in magnitude of the elevation factor:

1. EF < 1.0. The elevation factor is less than 1.0 at locations that are above sea level elevation (i.e., outside the GRS 80 ellipsoid).
2. EF = 1.0. For locations that are at approximately sea level (i.e., on the GRS 80 ellipsoid), the elevation factor is equal to 1.0.
3. EF > 1.0. At locations that are below sea level (i.e., inside the GRS 80 ellipsoid), the elevation factor is greater than 1.0.

12.3.4.3 Combination Factor

The combination factor is a dimensionless scale, or multiplication, factor that is used to convert a distance between points on the ground to an equivalent distance between the points as they are projected onto the state plane grid (i.e., on the Lambert conformal projection). By definition, the combination factor for a particular location is the product of the location's grid and elevation factors. The following equation represents the mathematical relationship between the combination, grid and elevation factors:

$$CF = (GF)(EF)$$

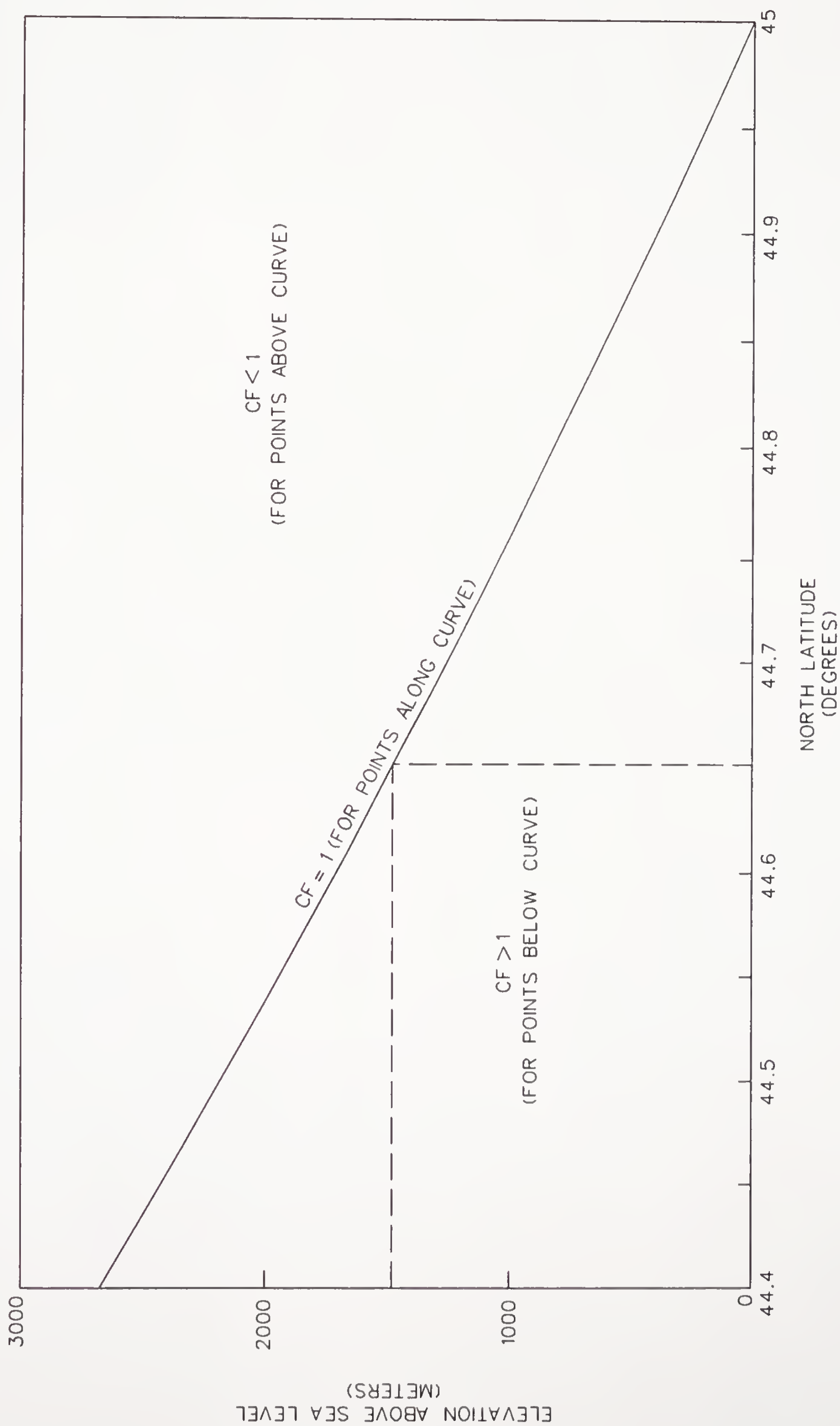
The range in magnitude of the combination factor at most locations in Montana (i.e., between 49° North and 45° North Latitudes) is less than 1.00000000 and greater than 0.99900000. A possible exception to this is that the combination factor may be greater than 1.0 at locations in the southwest tip of the State (i.e.,

below 45° North Latitude). In this region, the grid factor is greater than 1.0. If the elevation factor closely approximates 1.0, then the product of the grid and elevation factors may result in a combination factor that is greater than 1.0. Figure 12.3C provides a graph that illustrates this relationship. For example, at a location near 44.65° North Latitude that has an elevation of approximately 1500 m, the combination factor will approximate 1.0. Figure 12.3D presents a table that illustrates how the grid, elevation and combination factors vary according to location.

Location	Avon, MT	Lindsay, MT	Beartooth Pass, MT	Troy, MT	Monida Pass, MT
North Latitude	46° 35′54″	47° 13′13″	45° 00′17″	48° 27′52″	44° 33′20″
West Longitude	112° 35′31″	105° 09′02″	109° 24′36″	115° 53′44″	112° 16′00″
Elevation, m	1431.00	817.00	3121.00	575.00	2094.00
Grid Factor (GF)	0.999418616	0.999399187	0.999997169	0.999715546	1.000295660
Elevation Factor (EF)	0.9997754742	09998717992	0.9995104407	0.9999097696	0.9996714827
Combination Factor (CF)	0.9991942207	0.9992710633	0.9995076111	0.9996253413	0.9999670456

GRID, ELEVATION AND COMBINATION FACTORS FOR VARIOUS LOCATIONS IN MONTANA

Figure 12.3D



RELATIONSHIP BETWEEN ELEVATION AND COMBINATION FACTOR FOR LOCATIONS BELOW 45E NORTH LATITUDE

Figure 12.3C

12.4 GUIDELINES FOR BRIDGE DESIGN AND PLAN PREPARATION

This Section provides the designer with guidelines for working within the Montana State Plane Coordinate System during bridge design and plan preparation.

12.4.1 Survey Stationing and Plan Dimensions

In general, survey stationing for the Department's projects is employed on the state plane. However, the horizontal dimensions that are shown on the plans reflect actual ground dimensions. Project elevations are computed based on both state plane stationing and the vertical curve data provided by the Road Design Section. Although horizontal angles on the ground correspond to those on the state plane, lengths and areas require the application of the combination factor.

12.4.2 Application of the Combination Factor

The Department's projects are established with reference to control points on the state plane, and their construction plans and drawings contain the state plane coordinates and grid lines. The state plane coordinates of two ends of a construction line that appears on a plan may be used to compute the line's grid length and grid bearing or azimuth; however, the proper length to be used in design and construction is the line's actual length on the ground. The conversion between grid length and ground length can be achieved by employing the combination factor in one of two ways as follows:

1. Ground Distance to Grid Distance. To convert a ground distance to an equivalent distance on the state plane grid, multiply the ground distance by the combination factor.

2. Grid Distance to Ground Distance. To convert a distance on the state plane grid to an equivalent distance as measured on the ground, compute the length between the state plane coordinates and divide the resultant grid distance by the combination factor.

If the state plane coordinates of the corners of an area are known, the grid area encompassed within the defining boundary may be computed. Because this grid area does not correspond to the actual ground area, the difference is taken into account by employing the combination factor as follows:

1. Ground Area to Grid Area. To convert a ground area to an equivalent area on the state plane grid, multiply the ground area by the square of the combination factor.

2. Grid Area to Ground Area. To convert an area that is on the state plane grid to an equivalent area as measured on the ground, compute the area bounded by the state plane coordinates and divide the resultant grid area by the square of the combination factor.

To accommodate proper application of the combination factor without misunderstanding, a note is generally placed on each drawing within the plan set. This note gives explicit instructions to the user as illustrated by the following example:

Note: All distances shown on this plan are grid distances on the Montana State Plane Coordinate System. To obtain ground distances for laying out construction lines, divide grid distances by the combination factor 0.9998940.

The Photogrammetry and Survey Section provides the combination factor that is used in the note. The magnitude of the factor may differ among plan sheets depending on the length, orientation and location of the project as it is designed to convert between grid and ground distances for specific geographic locations.

12.4.3 Preliminary General Layout

The combination factor may be ignored in all preliminary engineering calculations leading up to the preliminary general layout. At this point,

all survey and mapping data should be on the state plane. The Hydraulics Section typically ignores the use of the combination factor in their analyses, and intersections and topography are referenced to state plane stationing.

Once the preliminary general layout has been determined, the method for preparing the final general layout will depend on the existence of site constraints at the substructure location. For when to apply the combination factor during the preparation of the final general layout, substructure locations may be categorized as either critical or non-critical as discussed in the Sections that follow.

12.4.4 Critical Substructure Locations

Critical substructure locations are where the substructure unit requires a precise location. Examples include urban jobs and railroad crossings where an accurate location of the substructure is necessary to accommodate the specific site and meet its dimensional and clearance requirements.

Precise locations of critical substructure stations are typically determined during preliminary engineering. During the preparation of the final general layout for a critical substructure unit, the following steps should be performed:

1. Fix the critical substructure stations that were determined during preliminary engineering.

2. Recompute actual span lengths by applying the combination factor.

12.4.5 Non-Critical Substructure Locations

Non-critical substructure locations are where the substructure unit does not require a precise location. Water crossings, as well as some urban and railroad crossings, are typically categorized as non-critical. Small deviations (i.e., less than 0.1 m) in the location of a non-critical substructure unit will generally have no significant adverse affect on the overall project.

During the preparation of the final general layout for a non-critical substructure unit, the following steps should be performed:

1. Fix the span lengths to a convenient number (e.g., multiples of 0.500 m).

2. Recompute the non-critical substructure stations using the combination factor.

3. Select one reference station to use for the structure. This is typically either the middle or one end of the structure.

4. Compute all non-critical substructure stations referenced from the fixed reference station by applying the combination factor to the actual span lengths.

12.4.6 Final General Layout and Footing Plan

The following guidelines are applicable to the development of the final general layout and footing plan:

1. Project Stationing. Compute and present all project stations as state plane stationing.

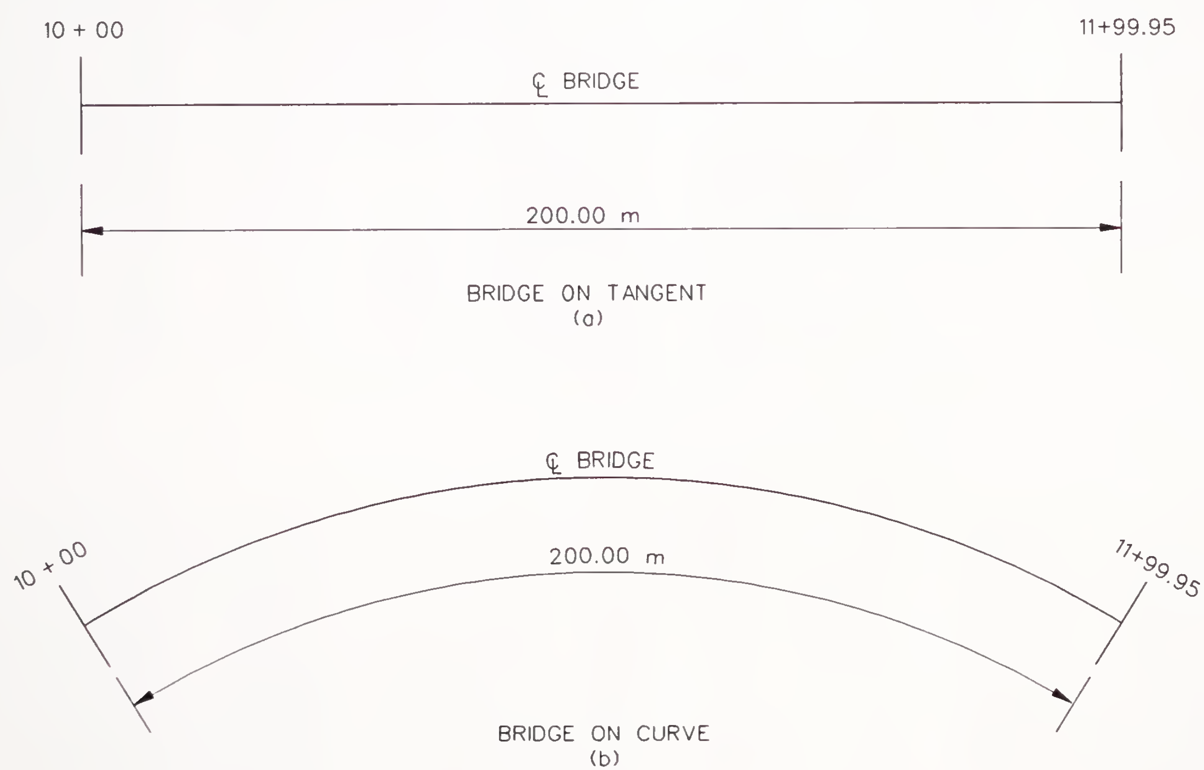
2. Curve Data. The vertical and horizontal curve data provided by the Road Design Section will be state plane distances and will be consistent with the data on the highway plans.

3. Structural Dimensions. Compute and present all dimensioned distances as actual ground distances, not distances on the state plane.

4. State Plane Coordinate System Note. Include the standard state plane coordinate system note on the final general layout. This note gives the combination factor that is applicable to the structure's site. See Section 12.4.2.

5. Total Length of Structure. Compute and present the total length of the structure along its centerline between the end stations. This

dimension shows that the actual ground length differs from the length of the structure as computed from the difference between its end stations (i.e., its length on the state plane grid). This is diagrammatically illustrated in Figure 12.4A.



PRESENTATION OF STRUCTURE LENGTH AND END STATIONS ON FINAL GENERAL LAYOUT

Figure 12.4A

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Chapter Thirteen
STRUCTURAL SYSTEMS AND DIMENSIONS

Chapter Thirteen provides guidance to MDT bridge designers in determining the most appropriate overall structure type and size to meet the structural, geometric, hydraulic, environmental and right-of-way characteristics of the site. This selection is a critical event in project development. The decision will significantly impact the detailed structure design phase, construction costs and maintenance costs over the life of the structure.

13.1 PROCEDURES

13.1.1 Scope of Work

To determine the proper application of Chapter Thirteen to a proposed project, it is necessary to identify the scope (or extent) of the proposed structural work. This will determine, for example, the MDT criteria for bridge width. Chapter 13 applies to a new bridge (i.e., construction of a new bridge at a new location) and to a bridge replacement (i.e., replacement of the entire existing bridge). See Chapter Twenty-two (Section 22.1) for definitions of project scopes of work on existing bridges.

13.1.2 Coordination

The structure type analysis will produce as an output the preliminary general layout and grade of the bridge for evaluation and further investigation by various units within the Department. These include the Road Design Section, Geotechnical Section, Environmental Services (e.g., for the Stream Preservation Act review), Right-of-Way Bureau and Utilities Section (for utilities and railroads). Ultimately, after finalization of the General Layout Sheet, the bridge designer will perform the detailed structural design of the bridge and prepare the final plans for project advertisement.

Chapters Two and Three describe the necessary coordination between the Bridge Bureau and other MDT units in selecting structure type and size.

13.1.3 Documentation

Chapter Four "Administrative Policies and Procedures" presents MDT's format and procedures for project documentation. The Preliminary Field Review (PFR) Report documents the characteristics of the project site. This, among other items, will be used to prepare the preliminary bridge layout and grade. Once the structure type and size has been approved, this will be documented in the Scope of Work Report. Section 4.1.2 presents the format and content of the Scope of Work Report.

13.1.4 Plan Preparation

Chapter Five presents MDT criteria and procedures for preparing bridge plans. This includes the sequence of construction plan sheets, CADD conventions and the content of individual sheets. One of these is the General Layout of Structure Sheet, including its plan view, elevation view and profile grade. The structure type analysis will yield the preliminary General Layout and Grade of the proposed structure. As practical, this preliminary sheet should be prepared according to the conventions in Chapter Five for final construction plans.

13.1.5 Preliminary Cost Estimate

Chapter Seven presents MDT criteria for producing construction cost estimates; Section 7.1 discusses preliminary cost estimates. At the structure type and size selection stage of project development, the cost estimate will typically be

based on the area of the proposed bridge deck (m^2), the type of structure, recent projects in the geographic area and engineering judgment. The bridge designer will include the preliminary cost estimate in the Scope of Work Report as discussed in Section 4.1.2.2.

13.2 GENERAL EVALUATION FACTORS

Section 13.2 provides a summary of the many evaluation factors which will impact the selected structural system and its dimensions with references, where applicable, to more detailed information.

13.2.1 Hydraulics

The Hydraulics Section will prepare a Hydraulics Report in advance of the Bridge Bureau's structure type and size selection. The critical hydraulic factors include:

1. channel bottom elevation and width;
2. water surface elevation for the design-year flood;
3. skew angle and side slopes of channel;
4. required low beam elevation if determined to be critical;
5. hydraulic scour potential;
6. passage of ice and debris; and
7. stream flow conditions.

See Section 13.7 for more discussion on bridge hydraulics.

13.2.2 Roadway Design

Often the considerations for bridge design are different than those for road design because the design life of the bridge is 3 to 4 times the design life of the roadway. Roadway design factors which impact structural type and size selection include:

1. horizontal alignment (e.g., tangent, curve, superelevation, skew);
2. vertical clearances and alignment (e.g., longitudinal gradient, vertical curves);
3. traffic volumes;
4. roadway width;
5. presence of sidewalks and bike lanes; and
6. clear zones through underpasses.

See Sections 13.5 and 13.6 and the **Montana Road Design Manual** for a detailed discussion on roadway design elements at bridges.

13.2.3 Structural

Section 13.3 provides information on the selection of a superstructure type, and Section 13.4 provides information on selecting a substructure/foundation type.

13.2.3.1 Constraints

The structural constraints which impact structure type selection include:

1. limitations on structure depth,
2. foundations and groundwater conditions,
3. anticipated settlement,
4. acceptable span lengths,
5. dead load restrictions,
6. seismic factors,
7. phased construction requirements,
8. scour,
9. anticipated lateral loads,
10. requirements for phase construction because of traffic, and
11. geometric limitations on configuration.

13.2.3.2 Number/Length of Spans

In general, the fewest number of spans as practical should be used. This will minimize the interference with the elements underneath the structure (e.g., railroads, highways, marine traffic, water passage). The minimum number of and length of spans will be determined by several factors including:

1. foundation conditions,
2. vertical clearance requirements,
3. waterway opening requirements,
4. safety of underpassing traffic,
5. navigation requirements,
6. ice and flood debris considerations, and
7. construction costs.

13.2.3.3 Structural Details

The use of bearings, drainage appurtenances, expansion joints, excessive skew, etc., is costly and creates construction and maintenance problems. The designer should attempt to select a structure type which will minimize the use of these structural details.

13.2.3.4 Seismic

It is important to address the seismic response of a structure during the preliminary structure type selection. The structural system should be selected in recognition of the importance of seismic response. Ideally, bridges should have a regular configuration so that seismic behavior is predictable and so that plastic hinging is promoted in multiple, readily identifiable and repairable yielding components. Selecting a structural form based solely on gravity-type loading considerations and then adding seismic-resistive elements and details is unlikely to provide the best solution.

All structures will be designed in accordance with the seismic provisions of the **LRFD Bridge Design Specifications**. Other analysis and design techniques may be warranted for retrofit or unique design projects. All bridges are required to meet at least a minimum level of seismic performance. Although the LRFD Specifications seismic provisions do not discuss preliminary structure type selection, certain guidelines should be followed. In general, structure type should be selected with the following considerations:

1. <u>Alignment</u>. Straight bridges are preferred because curved bridges can lead to unpredictable seismic response.

2. <u>Substructure Skew</u>. Substructure units should have little or no skew. Skewed supports cause rotational response with increased displacements.

3. <u>Superstructure Weight</u>. Superstructure weight should be minimized.

4. <u>Joints</u>. The deck should have as few expansion joints as practical, with a jointless deck being ideal.

5. <u>Foundations</u>. Shallow foundations should be avoided if the foundation material is susceptible to liquefaction.

6. <u>Substructure Stiffness</u>. Large differences in the stiffness of the substructure units should be avoided; i.e., the use of piers of uniform cross section with varying heights should be avoided if practical.

7. <u>Plastic Hinges</u>. Plastic hinges should be forced to develop only in the columns rather than the cap beams or superstructure and should be accessible for inspection and repair after an earthquake.

The Seismic Unit supports the Bridge Design Section in the seismic design and analysis of new and rehabilitated bridges. In this capacity, the Seismic Unit performs a significant amount of design work for bridges within the context of addressing seismic vulnerability. Bridge work which involves seismic concerns should be reviewed by the Seismic Unit.

13.2.3.5 Foundation Considerations

The following applies, in general, to foundation considerations:

1. Grade Adjustment. When considering structural-system selection, the ability to adjust the structure through jacking is an important issue. Jacking stiffeners or diaphragms may be required. The subgrade may settle differently from the calculated estimates. It is understood that, where superstructures and substructures are integral with each other, this facility for adjustment cannot exist.

   The nature of the subgrade should be considered prior to the final selection and design of the superstructure, substructure and foundation to ensure adjustability if needed.

2. Settlement Limits. Experience demonstrates that bridges can accommodate more settlement than traditionally allowed in design due to creep, relaxation and redistribution of force effects. Article 10.6.2.2.1 of the LRFD Specifications mandates that settlement criteria be developed consistent with the function and type of structure, anticipated service life and consequences of unanticipated movements on service performance. Further, in the commentary it suggests that angular distortions between adjacent spread footings greater than 0.008 in simple spans and 0.004 in continuous spans should not be ordinarily permitted.

13.2.4 Falsework

Temporary falsework is an expensive construction item. If the bridge is over a waterway and/or will have a high finished elevation, the cost of the falsework may become prohibitive, and the designer should consider another structural system.

The following will apply to the use of falsework:

1. Water. The use of falsework over water is generally discouraged. However, each specific site should be investigated on a cost versus risk basis. If a falsework opening for at least the five-year event can be provided, the use of falsework over water may be considered.

2. Railroads. Each railroad company has its own requirements for falsework over its facilities. Depending on the railroad company and the type and amount of railroad traffic, the railroad company may prohibit the use of falsework. The railroad company should be contacted early in project development to determine if falsework may be used and its minimum clearance requirements.

3. Traffic. Falsework may unduly interfere with traffic passing beneath the structure or may create an unacceptable safety hazard. For example, falsework may reduce the vertical clearance below acceptable levels and may require a barrier installation to shield traffic from the falsework. The designer should contact the District prior to using falsework over traffic. The minimum clearance for falsework will be determined on a case-by-case basis.

4. Environmental. Some sites may be very sensitive environmentally, and the use of falsework may be prohibited. Falsework for typical flat slab construction is usually not an issue.

Basically, if the use of falsework is unacceptable or impractical, this eliminates the use of any cast-in-place concrete or shored construction type structure.

13.2.5 Aesthetics

Structures should be aesthetically pleasing to the traveling public. The LRFD Specifications emphasize improving the appearance of highway bridges in the United States. It promotes uninterrupted lines, contours that follow the flow

of stresses and the avoidance of cluttered appearances. Structures should incorporate attractive shapes and surface treatments and should be similar in appearance to adjacent structures.

In addition to overall structure type, other factors will impact the aesthetic appeal of the structure (e.g., fine surface finish on concrete, color of steel). Aesthetic considerations may also become a factor in structure-type selection because, for example, of historic or local community considerations.

13.2.6 Environment

The evaluation of potential environmental impacts can have a significant impact on structure-type selection and configuration, especially for highway bridges over streams. See Section 13.8 for a detailed discussion on environmental considerations. The bridge designer must seriously evaluate these issues in project development.

13.2.7 Construction

13.2.7.1 General

The LRFD Specifications requires that, unless there is a single obvious method, at least one sequence of construction should be indicated in the contract documents. If an alternative sequence is allowed, the contractor should prove that stresses, which accumulate in the structure during construction, will remain within acceptable limits.

13.2.7.2 Access and Time Restrictions

Water-crossing bridges will typically have construction restrictions associated with their construction. These must be considered during structure type evaluation.

The time period that the contractor will be allowed to work within the waterway may be restricted by regulations administered by various agencies. Depending on the time limitations, a bridge with fewer piers or faster pier construction may be more advantageous even if more expensive.

Contractor access to the water may also be restricted. To work in or gain access areas, a work bridge may be necessary. Work bridges may also be necessary for bridge removal as well as new bridge construction.

13.2.7.3 Phase Construction

At times, due to the proximity of existing structures or a congested work area, it may be necessary to build a structure in several phases. The arrangement and sequencing of each phase of construction are unique to each project and due consideration must be given to requirements for adequate construction clearances and the requirements of the traveling public. If phase construction is required, then the phasing sequence and controlling lane/ construction dimensions must be shown on the plans.

The phase sequencing must be developed with the District. In general, it is usually allowable to restrict traffic to a 3.35-m lane width, although this depends on traffic speeds and District concurrence.

13.2.8 Construction Costs

Initial construction costs should be one factor in the selection of the structure type, but not the only factor. Future expenditures during the service life of the bridge should also be considered. The initial costs depend on a variety of factors including:

1. type of structure,
2. economy of design,
3. general state of the economy,

4. degree of occupancy and experience of local contractors,

5. vicinity of fabricating shops, and

6. local availability of structural materials and labor.

These factors may change rapidly, and the designer may have no control over them. It may be advisable to prepare competitive plans (i.e., for both concrete and steel superstructures) occasionally even for small-span structures. A review of the cost of structural components within a bridge, and that of contractor's claims, may direct the designer towards optimum combinations and the avoidance of future errors.

13.2.9 Highway Bridges Over Railroads

Railroad geometric requirements must be considered in structure-type selection. Chapter Twenty-one presents specific details which will apply to highway bridges over railroads.

13.2.10 Right-of-Way/Utilities

The Right-of-Way Bureau is responsible for securing project right-of-way. The designer should consider the following right-of-way factors when selecting the structure type:

1. Expensive Right-of-Way. If right-of-way will be expensive, this may lead to the use of reinforced earth abutments.

2. Structure Depth. The available right-of-way at the bridge site may affect the vertical alignment of the structure which may, in turn, affect the acceptable structure depth to meet the vertical clearance requirements.

3. Detours. For bridge rehabilitation projects, if right-of-way is not available for detours, it may be necessary to maintain traffic across the existing bridge.

Any bridge design must be consistent with MDT utility accommodation policies. Utility attachments to bridges are discussed in Chapter Fifteen.

13.2.11 Maintenance

The structure type selection will, over the life of the structure, have a major impact on maintenance costs. Based on type of material, the following is the approximate order of desirability from a maintenance perspective:

1. prestressed concrete,
2. reinforced concrete slab bridges,
3. unpainted weathering steel, and
4. painted structural steel.

With proper detailing and periodic simple flushing with water to remove corrosion-encouraging debris buildup, unpainted weathering steel can be relatively maintenance-free. Additional experience may move unpainted weathering steel up in the order of desirability from a maintenance perspective.

The following maintenance considerations apply:

1. Deck Joints. Open, or inadequately sealed, deck joints have been identified as the foremost reason for structural corrosion of structural elements by permitting the percolation of salt-laden water through the deck. To address this, the LRFD Specifications promotes jointless bridges with continuous decks, integral abutments and improvements in drainage. See Section 13.4.4 for a discussion on jointless bridges.

2. Re-bars. The overwhelming problem for reinforced concrete slabs is the corrosion of the re-bars, the volumetric expansion of corrosion products, and the resulting spalling and delamination of the concrete. Epoxy-coated reinforcement is judged to be the best solution currently available for use with steel re-bars.

3. Paint. The environmental concern for removing paint from steel structures makes the future use of painted steel structures questionable. Where practical, the application of self-protecting weathering steels should be used.

4. Drainage. Drainage facilities should be few and large, and elaborate plumbing systems should be avoided.

5. Bridge Inspection. In addition to the maintenance needs of the structure, the designer should consider the bridge inspection logistics.

6. Structural Details. As another maintenance/ inspection consideration, the designer should, as practical, limit the number of structural details (e.g., bearings, expansion joints).

13.2.12 Future Widening

In general, the designer should consider the possibility of future structure widening. For example, structures supported by single columns or cantilevered piers cannot practically be widened; a separate adjacent structure will be required.

Almost every superstructure type can be widened, but not with the same level of ease. Slabs, slab on girders, and systems consisting of prefabricated concrete elements lend themselves best to widening.

13.2.13 ADA

Many highway elements can affect the accessibility and mobility of disabled individuals. These include sidewalks, parking lots, buildings at transportation facilities, overpasses and underpasses. The Department's accessibility criteria must comply with the 1990 **Americans with Disabilities Act** (ADA) and with the **ADA Accessibility Guidelines for Buildings and Facilities** (ADA Guidelines). Where other agencies' or local codes require standards which exceed the **ADA Guidelines**, then the stricter criteria may be required. This will be determined on a case-by-case basis.

Chapter Eighteen of the **Montana Road Design Manual** presents the MDT criteria to comply with **ADA** for highway-related elements (e.g., sidewalks, parking lots, bus stops, curb ramps). Further, the **ADA** accessibility criteria depends on whether the sidewalk is on an accessible route or on a non-accessible route, as defined below. Where the Department is upgrading a facility and installing a sidewalk, the bridge must be configured to meet accessible route criteria:

1. Accessible Route. An accessible route is a continuous, unobstructed path connecting all accessible elements and spaces in a building, facility or site. A "site" is defined as a parcel of land bounded by a property line or a designated portion of a public right-of-way. A "facility" is defined as all or any portion of buildings, structures, site improvements, complexes, equipment, roads, walks, passageways, parking lots, or other real or personal property on a site. Interior accessible routes may include corridors, floors, ramps, elevators, lifts and clear floor space at fixtures. Exterior accessible routes may include parking access aisles, curb ramps, crosswalks at vehicular ways, walks, ramps and lifts.

   For highway projects, the application of the accessible route criteria applies to sites which are related to highway purposes. These include rest areas, recreational areas, park-and-ride lots, sidewalks adjacent to public roadways, etc. Most sidewalks adjacent to public highways are considered accessible routes.

2. Non-Accessible Route. A non-accessible route is any pedestrian facility which contains features that make it impractical to meet the criteria for accessible routes. These features include terrain that results in steep grades on the facility, narrow

sidewalks or stairs where adjacent development precludes widening or replacement with ramps. These features are typically associated with existing facilities; however, they may also affect the accessibility of routes on new construction projects.

The **ADA** criteria for sidewalks on a bridge are:

1. Longitudinal Slope. Provide the flattest longitudinal slope practical which, desirably, will not exceed 5%. Where slopes are greater than 5% on non-accessible routes, it will not be necessary to provide handrails, which are required on accessible routes.

2. Cross Slopes. Cross slopes greater than 2% may be used provided that transitions at cross slope changes are smoothly blended. The designer should attempt to provide the flattest cross slope practical.

3. Width. The minimum clear width at any isolated point along an accessible or non-accessible route shall be 915 mm. If the clear width is less than 1525 mm, then passing spaces at least 1525 mm by 1525 mm shall be located at reasonable intervals not to exceed 61 m. Note that, where the Department's typical 1.6-m sidewalk width is used (see Section 13.5), MDT meets all **ADA** requirements without the need for passing spaces.

Where necessary, the Bridge Bureau coordinates with the Human Resources Division, Civil Rights Bureau, to ensure compliance with the **Americans with Disabilities Act**. The Division will, for example, provide interpretations on the intent and application of the **Act**.

13.3 SUPERSTRUCTURES

This Section discusses those factors which should be considered in the selection of the superstructure type.

13.3.1 Superstructure Types/Characteristics

The following figures provide summary guidance in selecting the bridge superstructure type that is most appropriate for the given site conditions:

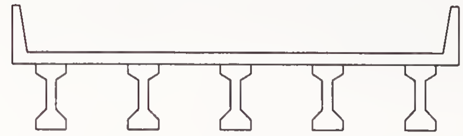
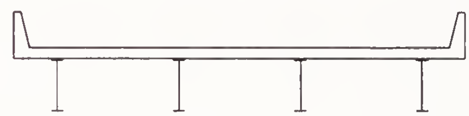
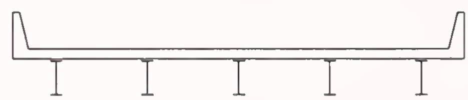
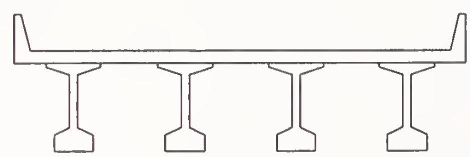
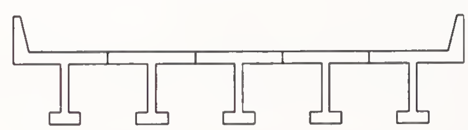
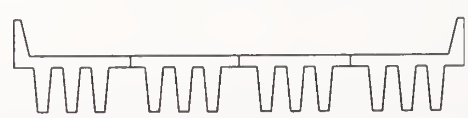
1. Past Usage of Bridge Types. Figure 13.3A presents the bridge types which have been constructed by MDT from 1985 to 1995.
2. Superstructure Types. Figure 13.3B presents a schematic cross section of those bridge types which are used in Montana. Each of these is further discussed in Section 13.3.2.
3. Span Lengths. Figure 13.3C indicates the typical ranges of span lengths for which the various superstructure types will generally apply.
4. Minimum Depths. Figure 13.3D presents the minimum depths for constant depth superstructures.
5. Girder Spacing Ranges. Figure 13.3E presents the range of girder spacings which typically apply to the superstructure types.
6. Superstructure Characteristics. Figure 13.3F tabulates several basic characteristics of the superstructure types in Figure 13.3B.

TYPE	NUMBER	PERCENTAGE
Prestressed, Precast Concrete I-Girders	108	47%
Reinforced Concrete Slabs	35	15%
Composite Rolled Steel Girders	26	11%
Jointed, Prestressed, Precast Longitudinal Concrete Elements	22	10%
Composite Steel Welded Plate Girders	21	9%
Other Types	19	8%
Total	231	100%

Dates: 1985 to 1995

NEW BRIDGE TYPES CONSTRUCTED IN MONTANA
(State-Owned Structures Only)

Figure 13.3A

Type	Structure Description	Cross Section Schematic
A	Prestressed, Precast Concrete I-Girders	
B	Reinforced, Cast-in-Place Concrete Slabs	
C	Composite Steel Welded Plate Girders	
D	Composite Rolled Steel Girders	
E	Post-Tensioned, Concrete I-Girders	
F1	Jointed, Prestressed, Precast Longitudinal Concrete Bulb Tees	
F2	Jointed, Prestressed, Precast Longitudinal Concrete Tri-Deck	

SUPERSTRUCTURE TYPES USED BY MDT

Figure 13.3B

Type	Structure Description	Subgroup	Span Length Ranges (in meters)				
			Up to 10	10-30	30-50	50-100	>100
A	Prestressed, Precast Concrete I-Girders	N.A.		x	x		
B	Reinforced, Cast-in-Place Concrete Slabs	Haunched	x				
C	Composite Steel Welded Plate Girders	N.A.		x	x	x	x
D	Composite Rolled Steel Girders	Straight		x			
E	Post-Tensioned, Concrete I-Girders	Straight		x	x		
F	Jointed, Prestressed, Precast Longitudinal Concrete Elements	1. Bulb Tees		x	x		
		2. Tri-Deck	x	x			

Note: See Section 13.3.2 for more precise span ranges.

SPAN LENGTH RANGES

Figure 13.3C

Type	Structure Description	Subgroup	Minimum Depth (Including Deck)	
			Simply Supported	Continuous
A	Prestressed, Precast Concrete I-Girders	N.A.	0.045 L	0.040 L
B	Reinforced, Cast-in-Place Concrete Slabs	Haunched	$\frac{1.2\,(S+3000)}{30}$	$\frac{S+3000}{30} \geq 165$ mm
C	Composite Steel Welded Plate Girders	N.A.	0.040 L	0.032 L
D	Composite Rolled Steel Girders	Straight	0.040 L	0.032 L
E	Post-Tensioned, Concrete I-Girders	Straight	0.045 L	0.040 L
F	Jointed, Prestressed, Precast Longitudinal Concrete Elements	1. Bulb Tees	0.045 L	0.040 L
		2. Tri-Deck	0.045 L	0.040 L

* *"Depth" refers to the total structure depth at the point of maximum positive moment, including the deck, where composite. "Span" is defined as the distance between center lines of bearings or the centerlines of piers where double bearings are present or the neutral axes of the vertical components where bearings are absent.*

Note: L = Span Length
S = Slab Span Length

MINIMUM DEPTHS*
(Constant Depth Superstructures)

Figure 13.3D

Type	Structure Description	Subgroup	Range of Girder or Web Spacing
A	Prestressed, Precast Concrete I-Girders	N.A.	1.5 m to 4.5 m
B	Reinforced, Cast-in-Place Concrete Slabs	Haunched	N.A.
C	Composite Steel Welded Plate Girders	N.A.	1.5 m to 4.5 m
D	Composite Rolled Steel Girders	Straight	1.5 m to 4.5 m
E	Post-Tensioned, Concrete I-Girders	Straight	2.5 m to 4.5 m
F	Jointed, Prestressed, Precast Longitudinal Concrete Elements	1. Bulb Tees	1.0 m to 2.0 m
		2. Tri-Deck	1.6 m to 2.6 m

GIRDER SPACING RANGES

Figure 13.3E

Type	Structure Description	Sub Group	For Skew	For Horz. Curve	Seismic*	Aesthetics	False Work Required?	Maintenance	Widening	Cost per Unit Deck Area (1999)
A	Prestressed, Precast Concrete I-Girders	N.A.	OK	OK	OK	OK	No	Good	OK	620
B	Reinforced, Cast-in-Place Concrete Slabs	Haunched	Good	Good	Good	Good	Yes	Good	OK	530
C	Composite Steel Welded Plate Girders	N.A.	OK	Good	Good	OK	No	**	OK	700
D	Composite Rolled Steel Girders	Straight	Good	OK	Good	Good	No	**	OK	650
E	Post-Tensioned, Concrete I-Girders	Straight	OK	No	Good	OK	No	Good	OK	—
F	Jointed, Prestressed, Precast Longitudinal Concrete Elements	1. Bulb Tees	Poor	Poor	OK	OK	No	Good	Good	590
		2. Tri-Deck	Poor	Poor	OK	OK	No	Good	Good	900

**Descriptions in column are for continuous structures. Simply supported structures are always poor.*

*** Good if weathering steel is used. Expensive (i.e., Poor) if painted.*

SUPERSTRUCTURE CHARACTERISTICS

Figure 13.3F

13.3.2 Superstructure Types

13.3.2.1 Type "A": Prestressed, Precast Concrete I-Girders

Because of their local availability, prestressed, precast concrete I-girders are the predominant type of superstructure used in Montana. MDT designations for prestressed, precast concrete I-girders for efficient design with their recommended maximum span lengths are as follows:

1. Type I — 17 m
2. Type MT-28 — 23 m
3. Type A — 26 m
4. Type IV — 35 m
5. Type M-72 — 45 m

Figure 13.3G illustrates a typical schematic cross section of prestressed, precast concrete I-girders. See the **MDT Bridge Standard Drawings** for details on the Department's typical designs.

The maximum spacing of girders is approximately 4.5 m. The slab overhang should be as shown in the **MDT Standard Drawings**.

The girders are supported by bearings on the substructure. The LRFD Specifications mandates the use of diaphragms at the supports. The **MDT Standard Drawings** illustrate diaphragm details. Variations from the standards requires approval of the Bridge Engineer.

Precast girders may be made continuous in the longitudinal direction for transient loads. In this arrangement, the girders retain their individual bearings, but their ends are incorporated in a common diaphragm which is cast monolithically with the slab.

13.3.2.2 Type "B": Reinforced, Cast-in-Place Concrete Slabs

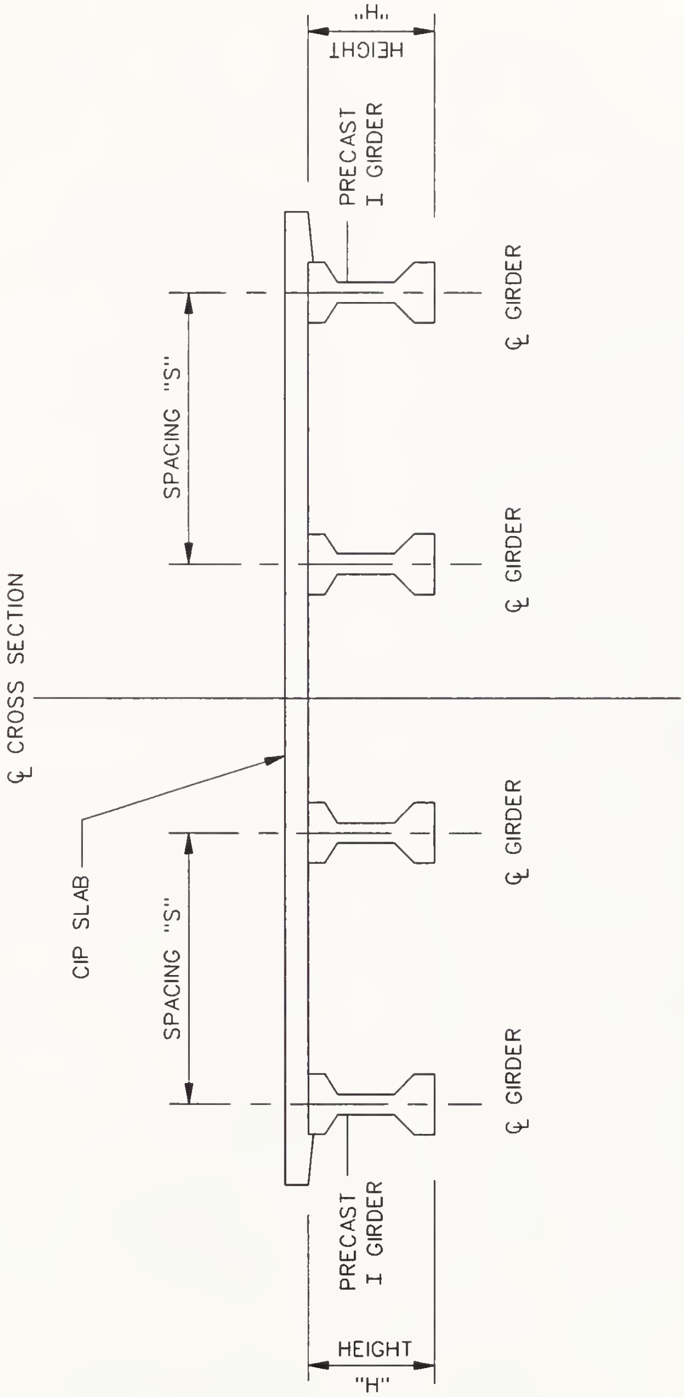
The reinforced, cast-in-place concrete slab is frequently used in Montana because of its suitability for short spans and low clearances and its adaptability to skewed and curved alignments. It is the simplest among all superstructure systems, it is easy to construct, and structural continuity can be achieved without difficulty. Its use is conditional, however, upon the local availability of ready-mix concrete.

For maximum span lengths, MDT uses 7.5 m for simply supported slabs and 10.5 m for continuous slabs. Figure 13.3H depicts the elevation and plan schematic of a three-span, continuous, haunched slab bridge with a significant skew.

In general, haunching is used to decrease maximum positive moments in continuous structures by attracting more negative moments to the haunches and by providing adequate resistance at the haunches for the increased negative moments. It is a simple, effective and economical way to maximize the resistance of thin concrete slabs. Typical MDT practice is to use the straight haunch.

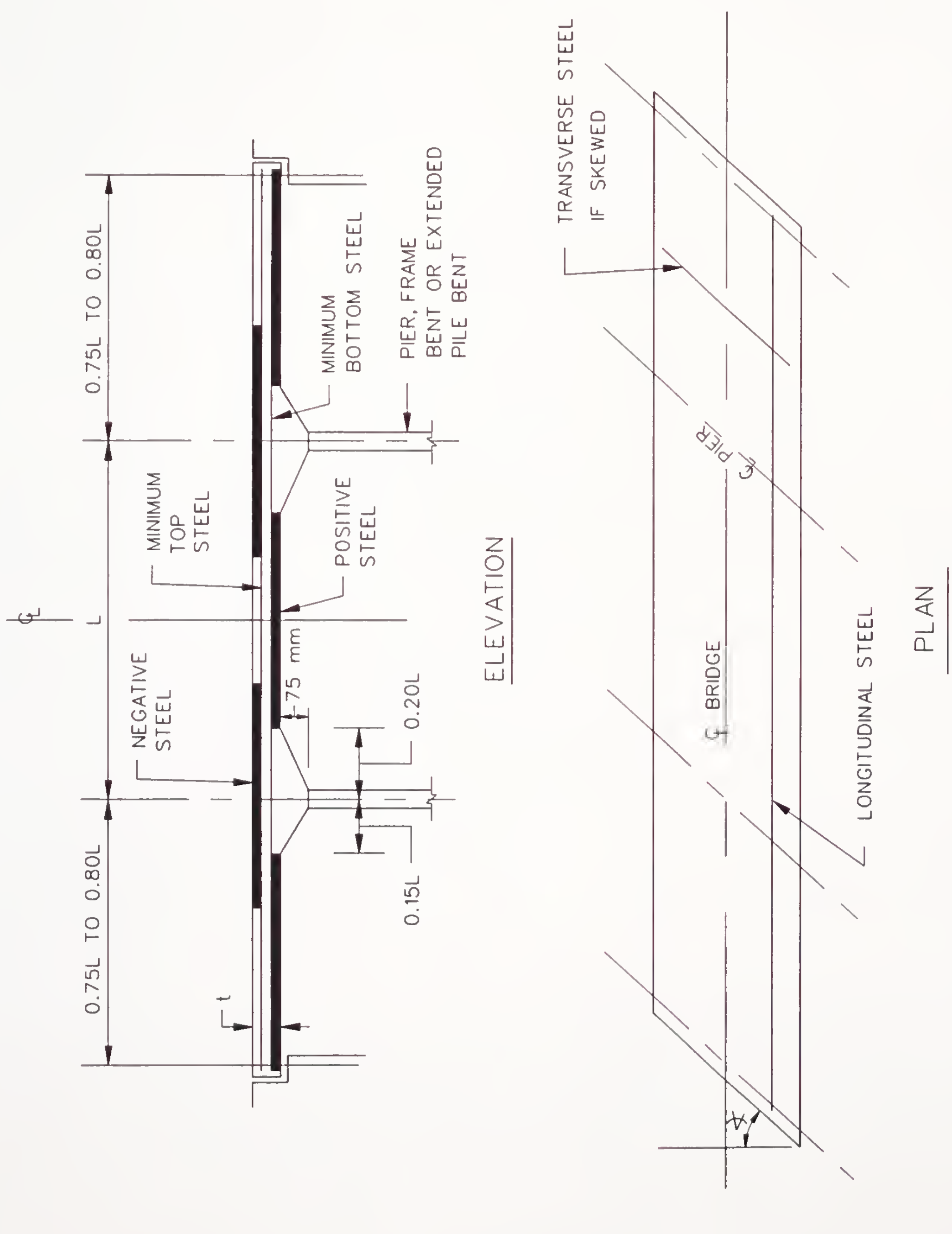
The preferable ratio between end and internal span is approximately 0.75 to 0.80 for economy, but the system permits considerable freedom in selecting span ratios. Except for architectural reasons, the length of the haunch need not exceed the value of 0.20L indicated in Figure 13.3H, where "L" is the length of the center span; longer haunches may be unnecessarily expensive and/or structurally counterproductive.

For interior piers with low height, narrow concrete walls or extended pile bents are recommended; otherwise, a concrete bent is practical. If the wall reinforcement is extended at the top, the deck could be constructed monolithically with the walls. For end supports, the MDT standard stub end bent should be used, resulting in a jointless deck structure.



COMPOSITE SLAB WITH PRESTRESSED, PRECAST AASHTO I-GIRDERS
(Type A)

Figure 13.3G



TYPICAL REINFORCED CONCRETE SLAB
(Type B)

Figure 13.3H

13.3.2.3 Type "C": Composite Steel Welded Plate Girders

MDT typically limits the use of structural steel girder superstructures to longer spans (45 m ≤ L ≤ 85 m), difficult geometrics (e.g., horizontal curves) or limited vertical clearances (e.g., for passage of ice and debris). Figure 13.3I illustrates a typical section for a composite steel welded plate girder superstructure.

Steel plate girders are designed to optimize weight savings and fabrication and erection costs. Top flanges of composite plate girders are typically smaller than their bottom flanges. The flange section is varied along the length of the bridge to save cost by offsetting the increased fabrication costs of welded flange transitions with larger savings in material costs. The most economical location for a flange transition is commonly at a field splice. Typically, only flange thicknesses, not widths, are varied within a field section. The webs of plate girders are typically deeper than the webs of rolled beams. To save in total costs, sometimes web thicknesses are increased to avoid the use of stiffeners.

Due to buckling considerations, the stability of the compression flange (i.e., the top flange in positive-moment regions and the bottom flange in negative-moment regions) must be addressed by providing lateral-brace locations based upon a simple calculation instead of the traditional 7.5-m diaphragm spacings of the former Standard Specifications for Highway Bridges.

On straight bridges (skewed or non-skewed), diaphragms are detailed as secondary members. On curved bridges, diaphragms must be designed as primary members, because curved girders transfer a more significant amount of load between girders through the diaphragms.

13.3.2.4 Type "D": Composite Rolled Steel Girders

Steel girders are characterized by doubly symmetrical as-rolled cross sections with equal-dimensioned top and bottom flanges and relatively thick webs. Thus, the cross sections are not optimized for weight savings but are cost effective due to lower fabrication and erection costs. The relatively thick webs preclude the need for web stiffeners. Unless difficult geometrics or limited vertical clearances control, rolled steel girder superstructures are most cost effective in rather short spans, with a maximum limit of approximately 40 m.

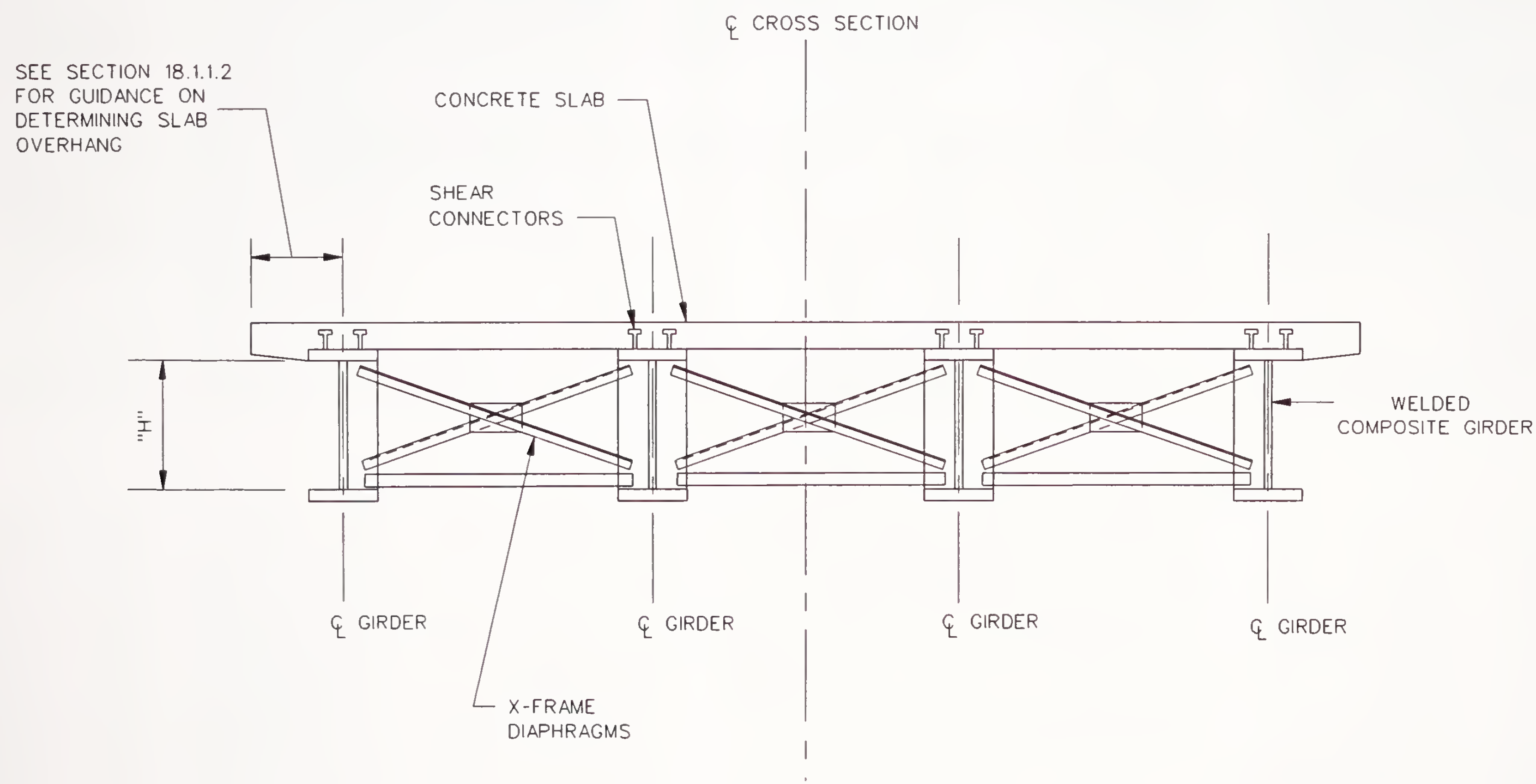
Rolled steel beams are available in depths up to 1 m, with the beams above typical depths rolled less frequently at intervals of about 16 weeks. Before beginning final design, verify with one or more potential fabricators that the section size and length are available.

On straight bridges (skewed or non-skewed), diaphragms are detailed as secondary members. On curved bridges, diaphragms must be designed as primary members, because curved girders transfer a more significant amount of load between girders through the diaphragms.

13.3.2.5 Type "E": Post-Tensioned, Concrete I Girders

The primary advantage of post-tensioned, concrete I-girders, when compared to the traditional prestressed, precast concrete girder (Type A), is its heightened level of structural continuity. The Type A superstructure can be designed to provide continuity for live-load only. For a Type E design with single-stage post-tensioning, the draped post-tensioning cables provide counterforce effects and general continuity. For a Type E design with two-stage post-tensioning (first stage before deck placement and second stage after), the design eases construction and enhances the economy of the single-stage continuity.

Designers should consider the use of post-tensioned I-girder superstructures where the enhanced redundancy of a truly continuous girder is desired or economy dictates. The span length range for Type E is 18 m to 50 m.



TYPICAL CROSS SECTION FOR COMPOSITE STEEL WELDED PLATE GIRDERS (Type C)

Figure 13.3I

Because the upper limit on span length for both Type A (M-72) girders and Type E post-tensioned bulb-tee girders is 50 m, these two types should be directly compared based upon total economy. In a Type E girder, the post-tensioning tendons are a replacement for many of the prestressing tendons in the M-72 girder.

The designer should consider, the limits of practical hauling from the plant to the job site when selecting the span length and girder type and size. Fabricators should be contacted early in project development for information regarding the availability and feasibility of hauling to a specific site.

Figure 13.3J illustrates a schematic of a post-tensioned concrete I-girder.

13.3.2.6 Type "F": Jointed, Prestressed, Precast Longitudinal Concrete Elements

MDT will only typically use the jointed, prestressed, precast longitudinal concrete superstructure (bulb tee or tri-deck) at sites which meet all of the following conditions:

1. bridges not maintained by the State;
2. remote sites;
3. low clearances;
4. simple geometrics (tangent sections only, skew ≤ 15°);
5. span length ≤ 35 m for bulb tee sections 1.195 m deep, span lengths ≤ 25 m for bulb tee sections 0.890 m deep; for rib decks consult with local producers to determine appropriate span limits; and
6. single-span bridges are the preferred layout for bulb tees because of the difficulty in obtaining a suitable ride, although multi-span tri-deck layouts do not seem to have particular problems with ride quality because of their shorter spans.

MDT normal practice is to use only simply supported designs for Type F. Figure 13.3K illustrates a schematic cross section for this superstructure type.

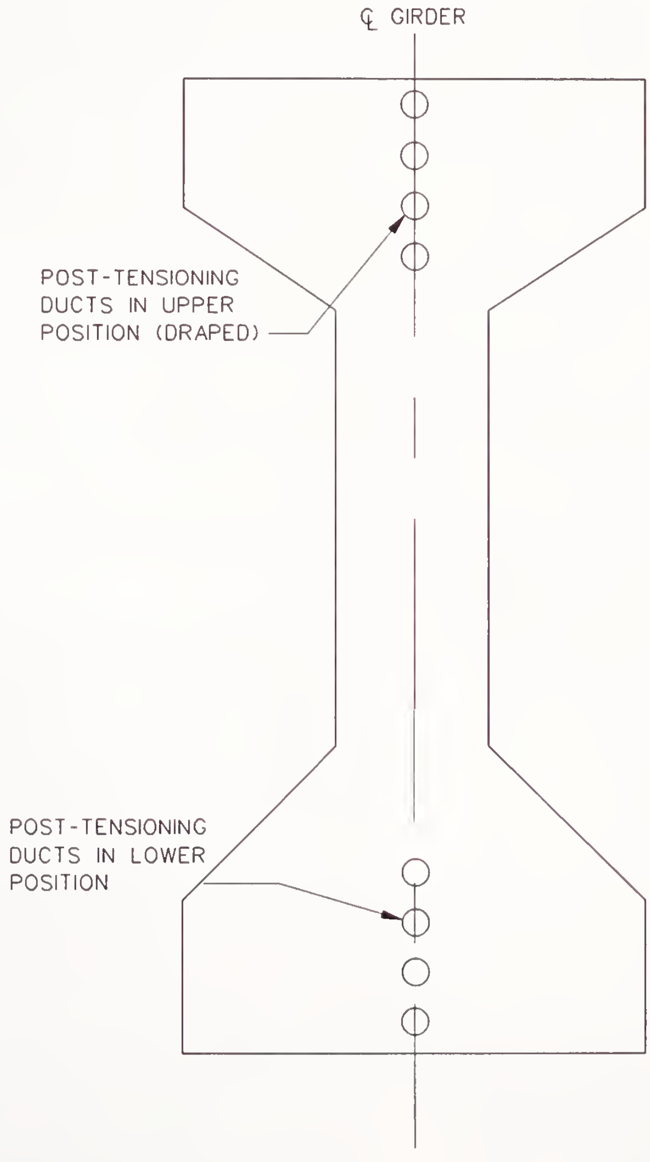
Prestressed, precast concrete longitudinal elements of various cross sections have been used without transverse post-tensioning to create a bridge deck. The long-term performance of these deck systems under heavy traffic is a concern because of possible disintegration and deterioration of longitudinal shear keys between the elements. This could result in potential overloading of the elements and maintenance problems for the deck, which is the reason that these types of bridges are not used on State-maintained facilities.

Typical MDT practice is as follows:

1. Precast elements are designed for a future overlay of 40 mm of plant mix.
2. A traditional trapezoidal key is used with a 10-mm gap between elements.

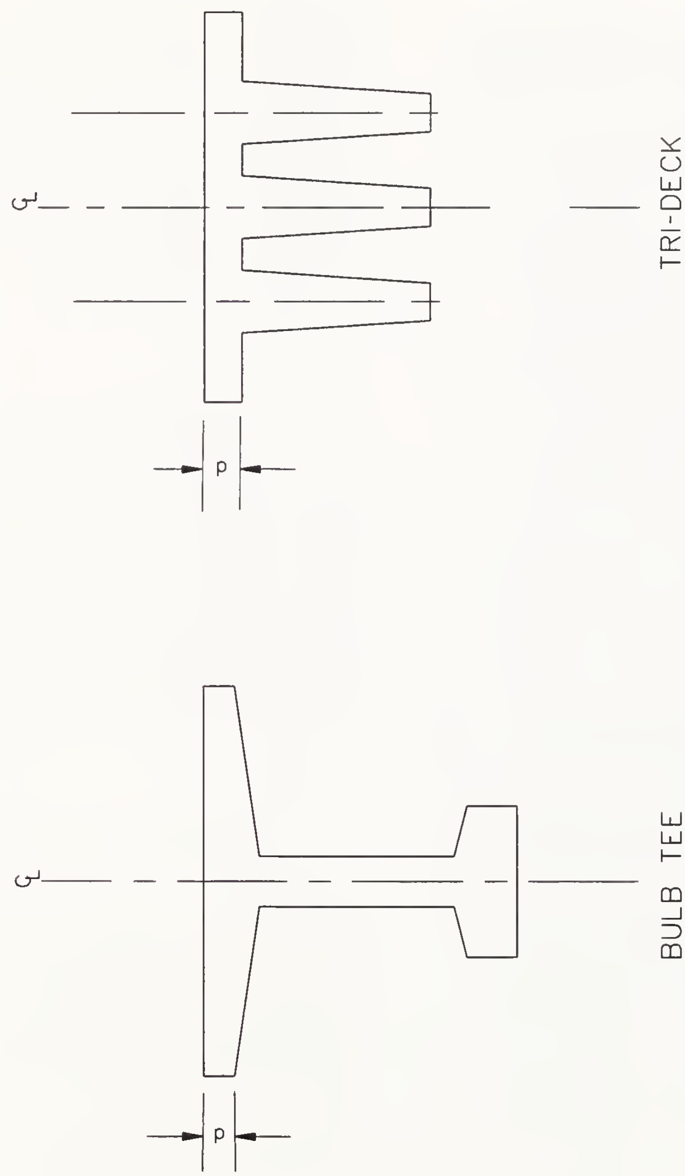
As illustrated in Figure 13.3K, MDT uses two basic precast, prestressed concrete elements that can be assembled into simply supported deck systems economically. The basic idea is that the precast elements either serve as the finished roadway or provide an uninterrupted formwork for a structural concrete overlay.

No variation of this system is applicable to curved structures. Skew is possible, but forming and casting the ends of the element with an angle other than 90° will cause some deterioration of ride quality and difficulty in manufacturing. The desirable limit for skew is 15°; skew angles greater than 30° are not allowed. This system can also be made continuous in the longitudinal direction by using a monolithic diaphragm and continuity steel or by using longitudinal post-tensioning similar to precast concrete girders. Tri-decks, however, lack an effective bottom flange, and they need special measures to improve the compressive strength of the stems at the point of junction.



POST-TENSIONED, CONCRETE I-GIRDER
(Type E)

Figure 13.3J



TYPICAL PRECAST CONCRETE ELEMENTS
(Type G)

Figure 13.3K

Attention must be provided to the treatment of the roadway crown when using this type of element. The order of preference for the MDT is:

1. no crown (deck will be flat);

2. constant and uniform crown across deck; and

3. crown break at a joint between longitudinal elements; the crown break cannot be placed on the top of an element.

13.4 SUBSTRUCTURES AND FOUNDATIONS

13.4.1 Objective

This Section discusses several types of substructure and foundation systems used by the Department, and it presents their general characteristics. The designer should use this information to select the combination of substructures and foundations which are suitable at the site so that the proposed elements will satisfy economically the geometric requirements of the bridge and to safely use the strength of the soil or rock present at the site to carry the anticipated loads. Chapter Nineteen discusses the detailed design of substructure elements, and Chapter Twenty discusses the design of foundations.

13.4.2 Substructure vs. Foundation

The demarcation line between substructure and foundation is not always clear, especially in the case of extended pile bents and drilled shafts. Traditionally, foundations include the supporting rock or soil and parts of the substructure which are in direct contact with, and transmit loads to, the supporting rock or soil. In this **Manual**, this definition will be used.

13.4.3 Substructure vs. Superstructure

A similar difficulty exists in separating substructure and superstructure where these parts are integrated. This **Manual** will refer to the bearings and any component or element located above the bearings as the superstructure.

13.4.4 Jointless Bridges

When practical, a jointless bridge should be considered in design. Problems with expansion joints include corrosion caused by deicing chemicals leaking through the joints, accumulation of debris and other foreign material restricting the free joint movement often resulting in joint damage, differential elevation at the joints causing additional impact forces, unexpected bridge movements and settlements that affect the joint, and high initial and maintenance costs.

Joints can be eliminated with special consideration to:

1. load path,
2. gravity and longitudinal loads,
3. effects of concrete creep and shrinkage and temperature variations,
4. stability of superstructure and substructure during construction and service,
5. skew and curvature effects,
6. the superstructure - abutment - foundation connection design and details,
7. effects of superstructure and substructure stiffness,
8. settlement and earth pressure,
9. effects of varying soil properties and type of foundation, and
10. effect of approach slab and its connection to the bridge.

Jointless bridges in service have demonstrated the ability to perform under the previous considerations. Therefore, in the absence of in-depth analyses, it is reasonable to design a jointless bridge under the following parameters. Exceeding one or more of these parameters will require a more detailed analysis:

1. 50 mm of total movement at the abutment,
2. 20 degree skew or less, and
3. abutment types that are flexible with one row of piles.

13.4.4.1 Load Path

The load path can be described by how a load is distributed from its point of application through the bridge system. This distribution depends upon the relative stiffness of the individual bridge elements and the boundary conditions. It is important to consider the bridge as a complete system that resists the loads applied to it rather than to consider the bridge elements as resisting loads independently of the other elements. For example, the load path for an ice load applied to an intermediate bent includes the superstructure and the other bents of the bridge. The relative stiffness of the superstructure and the bents determine how the ice load is distributed. In most cases, it is conservative to assume the intermediate bent resists the full ice load and no load is distributed to any other elements. However, in some load situations, the distributed load may overstress another element that was not assumed to resist any load.

A longitudinal load path may consist of a superstructure segment transferring all the load to one bent while the other bents are released from loads with expansion bearings. Another load path may consist of transferring the load to all the bents through fixed connections.

Bridges that are symmetrical will respond better to loads. Bridges that are unsymmetrical will have high stress points.

Consideration of the load path due to thermal movement must be investigated to ensure that the actual response of the structure matches the expected response.

13.4.4.2 Longitudinal Loads

Longitudinal loads can be categorized as either internal or external forces. Internal forces such as thermal and shrinkage must be released. External forces such as braking, soil pressure and seismic must be resisted. Because design requirements for internal forces are opposite external forces, it is difficult to satisfy both conditions. Joints and expansion bearings or flexible supports release internal forces, and continuous decks, fixed connections and rigid supports resist external forces.

Bridges with joints and expansion bearings are not very efficient in resisting external loads. Typically, abutments must be constructed to resist backwall pressure or the joints may close. Intermediate bents with expansion bearings become unsupported longitudinally and require additional strength to react as a cantilever. Finally, intermediate bents with fixed bearings resist larger longitudinal loads and require additional strength.

A jointless bridge is efficient in resisting external loads because the load is shared by all bents, and the backwall pressure is resisted by the opposite bridge end. Intermediate bents are supported by the superstructure and abutments so that they do not act like cantilevers. A jointless bridge can be designed to balance the internal force effects with the external force effects. This can be achieved by balancing the longitudinal stiffness required to resist external loads with the stiffness required to release internal loads.

Internal forces in a bridge can be analyzed by segregating the bridge into segments. Each segment is bounded by an expansion joint or the end of the bridge. Internal forces originate from the center of the longitudinal stiffness of a segment, which is not necessarily the center of that segment.

13.4.4.3 Longitudinal Bridge Stiffness

Conceptually, stiffness can be thought of as a spring and is defined as the force required to produce a unit deflection.

Longitudinal bridge stiffness or the stiffness parallel to traffic is a function of the superstructure, bearings or connections, substructure, skew, foundation and soil properties. These variables should be considered when analyzing the longitudinal response of a structure.

Generally, the superstructure axial stiffness is so large that it can be considered rigid compared to the other variables that affect longitudinal stiffness. Therefore, the superstructure will translate before it compresses.

Each connection has six degrees of freedom, which may be released, fixed or partially fixed. The connection can control the continuity of the load path and the stiffness of the overall bridge system. Different connections provide many options to resist or release forces in any degree of freedom. Typically, connections are assumed to be released or fixed, but the actual response may be somewhere in between. For example, an expansion bearing is assumed to be released but, due to friction, will provide some resistance. Partially fixed connections, such as elastomeric bearings, have some advantages that can help balance the loads throughout the bridge system. A partially fixed connection is designed to flex or give so that the load applied to the connected element is limited or reduced.

The substructure stiffness can be expressed as:

$$\alpha EI / L^3$$

Where:

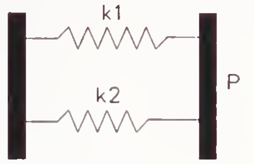
α = determined by the boundary conditions
E = modulus or elasticity
I = moment of inertia
L = effective member length

The boundary conditions are controlled by the connection of the elements. For the ideal fixed-fixed column, α is equal to 12 and, for a fixed-pin column, α is equal to 3. Where ideal conditions are not present because the cap or foundation is not completely fixed, α is somewhere between 12 and 3. The modulus of elasticity is determined from the compression strength which is defined by a specific 28-day strength. Because the actual 28-day strength typically exceeds the specified strength by 20% to 25% and concrete continues to gain strength with age, the actual strength and corresponding modulus are higher. Tests have shown the actual strength to be 1.5 to 2.7 times higher than the specified strength. This should be reflected by increasing the modulus of elasticity by 1.5. For concrete members, the moment of inertia is dependent on whether the concrete is cracked or uncracked. To determine maximum forces, it is conservative to use the gross moment of inertia. To calculate maximum displacements, an effective moment of inertia should be used. It may be too conservative to use the gross moment of inertia to analyze internal forces. The effective member length is the flexible length of the member, not the length based on centerline-to-centerline geometry. The effective member length can be different in the longitudinal direction than the transverse direction. If the substructure is expected to displace under thermal movement, then it should be designed to be flexible and to accommodate the required movement.

When a bent is skewed, the longitudinal bridge stiffness mobilizes the bent longitudinal and transverse stiffnesses. Typically, the transverse bent stiffness is much higher than the longitudinal bent stiffness. Therefore, the longitudinal bridge stiffness is increased. This increase can be reduced by using a single round column bent that has the same longitudinal and transverse stiffness.

Foundations are the structural elements that transfer vertical, lateral and rotational loads into the soil by soil-structure interaction. Soil-structure interaction is influenced by the type and geometry of the foundation and the characteristics of the surrounding soil. In typical designs, the foundation is considered to be infinitely stiff. However, the foundation stiffness should be compared to the substructure stiffness to verify this assumption. In many cases, the longitudinal bridge stiffness will decrease if the foundation stiffness is combined with the substructure stiffness. Foundation types should be matched with the intent of the design. If the foundation is considered rigid, then it should be very stiff. If the foundation is intended to accommodate translation, then it should be flexible.

These individual stiffness variables combine in parallel or series to form an equivalent longitudinal spring. To differentiate between parallel and series, consider the displacement of the two springs. If two springs have the same displacement, then they are in parallel:



If two springs have relative displacements, then they are in series:

13.4.4.4 Skew

Thermal expansion of a jointless superstructure causes compression of the backfill at the abutments. These passive earth forces act on both ends of the jointless superstructure (Figure 13.4A). The forces are eccentric and will tend to rotate the superstructure. This rotational force is resisted by transverse backwall-soil interaction and the abutment substructure. Soil-backwall interaction is considered adequate to resist the superstructure rotation for skew angles less than 15 degrees. For skew angles greater than 15 degrees, additional resistance must be provided by the abutment substructure.

13.4.4.5 Geotechnical Considerations for Jointless Bridges

13.4.4.5.1 Site Conditions

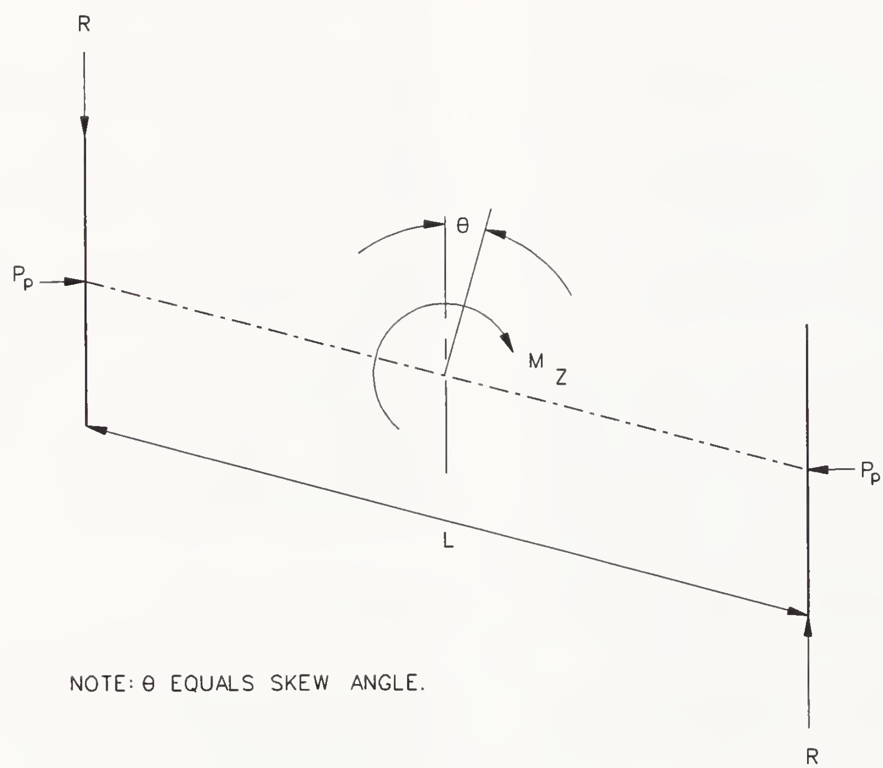
Geotechnical considerations for jointless bridges are generally the same as for jointed bridges, with additional consideration given to soil structure interactions and longitudinal stiffness.

Subsurface soil conditions at the bridge site must be determined to an acceptable degree prior to determining if a jointless bridge is a practical option. Consult with the Geotechnical Section regarding subsurface conditions at the bridge site. Generally, geotechnical investigations consistent for jointed bridges are acceptable for jointless bridges, with special consideration given to the upper strata where longitudinal foundation movements are expected. Jointless bridges require superstructure movements to be accommodated by the foundation through soil/structure interaction. This requires "flexible" foundations that are capable of deflecting longitudinally without damage to the foundation or significant disturbance to the roadway surface. Soil conditions that allow the use of flexible foundations range from soft clay to loose sand. Generally, "poor" soil conditions result in foundations that have less lateral stiffness and accommodate larger lateral movements than "good" soil conditions. Dense sand and gravel or hard clays will often result in foundations that are too stiff and cannot deflect horizontally the amount required by long superstructures.

Softer soil conditions have a greater potential for settlement problems. Thus, soil conditions that favor longitudinal movements of the foundations also have an increased risk of significant axial settlement. While longitudinal deflections of the foundations are encouraged, axial settlement is discouraged. Foundations at jointless bridges will generally require a more in-depth geotechnical analysis than jointed bridges.

13.4.4.5.2 Foundation Type

Foundations are the structural elements that transfer vertical, lateral and rotational loads into the soil by soil-structure interaction. Soil-structure interaction is influenced by the type and geometry of the foundation and the characteristics of the surrounding soil. In typical designs, the foundation is considered to be infinitely stiff. However, the foundation stiffness should be compared to the substructure stiffness to verify this assumption. In many cases, the longitudinal bridge stiffness will



ROTATION DUE TO THERMAL EXPANSION

Figure 13.4A

significantly decrease if the foundation stiffness is combined with the substructure stiffness. Foundation types should be matched with the intent of the design. If the foundation is considered rigid, then it should be very stiff. If the foundation is intended to accommodate translation, as is typical in jointless bridges, then it should be flexible.

Deep foundations such as piles are typically very stiff axially, but flexible laterally. Thus, they are good choices for jointless bridges, under certain soil conditions. Drilled shafts are similar to piles, except they are typically stiffer laterally and are generally used in more competent soil stratifications. Shallow foundations such as spread footings are generally very stiff both axially and laterally, especially at the embedment depths typically used for bridge foundations.

The type of foundation used at a jointless bridge will largely be determined by the type of soil at the project site. It will also be a function of the length of the span or lateral movement expected. Thus, the soil conditions can control the foundation type, which can control the length of span used for a jointless bridge. Under certain soil conditions, jointless bridges are not practical.

Very loose to loose cohesionless soil or very soft to soft clay will require deep foundations such as piles or, possibly, drilled shafts. Deep foundations in these soil deposits will be flexible enough to accommodate large longitudinal movements associated with long jointless bridges. However, high to moderate axial settlement can be expected in these soils. Extra conservatism may be warranted when designing the foundations to resist axial loads to limit settlement.

Dense cohesionless soils such as sandy gravel, gravelly sand, or cobbles and boulders are often not well suited for deep foundations due to the difficulty in locating the piles or drilled shafts to the required elevations to resist the axial loads. When deep foundations are used in these cases, the lateral stiffness is usually very high and typically will not permit the longitudinal movements necessary for a long jointless bridge. Short jointless bridges, however, may use these types of foundations in these soil deposits, when significant longitudinal movement is not expected. Typically, spread footings can be used in these soil conditions. Spread footings are typically very stiff foundations and can often be considered rigid in soil deposits of this nature. Axial settlements in these soil deposits are likely to be negligible.

Medium dense cohesionless soil or stiff to hard clays are intermediate materials. Deep foundations or shallow foundations can both be used in these materials. However, the stiffness of shallow foundations such as spread footings is likely to be too high to be used on a long jointless bridge. Even deep foundations such as piles, which are usually flexible, are likely to be too stiff to allow enough longitudinal movement for a moderate to long jointless bridges. In these soil conditions, the length of the bridge and the foundation type will be critical. Axial settlements may range from negligible to moderate in these soil deposits.

13.4.4.5.3 Abutments

Backfill limits and in-situ soil conditions should be considered very carefully when examining longitudinal abutment movements. Granular backfill consisting of crushed sand and gravel can mobilize full passive pressure at wall movements of approximately ½% - 1% of the height. If abutment movements are expected to achieve full passive pressure of the backfill, the capacity of the structure to move at, and tolerate, full passive pressure needs to be verified. The passive pressure of both the backfill and the in-situ material should be calculated using log spiral methods that incorporate wall friction such as the method developed by Caquot & Kerisel. Abutment movement necessary to develop full passive soil pressure can be estimated using data provided by Clough and Duncan, 1991. The limits of the backfill should be adjusted based on the expected failure surface for the two soils, the expected longitudinal

movement, and the amount of passive pressure that can be tolerated by the structure.

Pile stiffness should also be incorporated into the analysis of abutments. The types of piles used at the abutments will depend largely on the in-situ soil conditions and the intent of the design. The Geotechnical Section should be requested to make recommendations regarding pile type and length based on the subsurface investigations. However, it should be noted that flexibility at the abutments is required, especially when integral abutments are used. For this reason, where soil conditions permit, H-piles should be used with integral abutments. Pipe piles may also be acceptable. Special features at the upper portions of the pile may be required to achieve the desired flexibility.

Select backfill should be installed at bridge abutments, where recommended by the Geotechnical Section.

Additional drainage features such as weep holes, drainage mats and drain pipe should be considered for use in areas of cohesive soil or frequent inundations.

13.4.5 Integral Abutments

Integral abutments are end bents where the superstructure is extended directly into the abutment backwall. There is no joint in the bridge deck, the backwall is rigidly connected to the pile cap, and there are no bearings under the beams. Integral abutments require flexible foundation elements to allow superstructure rotation and thermal motion. Typically, a single row of piles will provide the required flexibility, but drilled shafts or spread footings will not.

Integral abutments will create negative moments in the slab-abutment zone. Additional reinforcement details are required.

Integral abutments are limited by the translational movement of the piles. There can not be any settlement in the piles because the superstructure can not be raised for maintenance. If there is a possibility of settlement, consider a semi-integral abutment.

13.4.6 Semi-Integral Abutments

Semi-integral abutments are similar to integral abutments except there is a pinned connection between the backwall and the pile cap and the beams rest on a bearing. This is MDT's typical stub abutment configuration. Expansion bearings can be used to reduce translation in the substructure. The pin joint between the backwall and the cap allows the superstructure to be raised if needed.

13.4.7 Substructures (Intermediate Supports)

13.4.7.1 Type Selection

The following summarizes MDT typical practice for the selection of intermediate supports for bridges:

1. Highway/Water Crossings. If lateral forces (e.g., seismic, debris, ice) are light, an extended pile bent is the preferred selection for the intermediate support. Otherwise, use a solid shaft pier or drilled shaft depending on soil conditions.

2. Highway/Meandering Streams. Use single round columns with hammerhead caps.

3. Highway/Highway Crossings. Use multi-column bents with caps.

4. Highway/Railroad Crossings. Use multi-column bents with caps or a single column pier with a hammerhead cap and a crash wall, if required by railroad clearance policies.

The following sections briefly describe the intermediate supports used by MDT.

13.4.7.2 Extended Pile or Shaft Bents

Under certain conditions, the economy of substructures can be enhanced by extending the deep foundation above ground level to the superstructure. These conditions exclude the presence of large horizontal forces which may develop due to seismic activity, collision by vehicles, ice, or stream flow intensified by accumulated debris.

The extended piles or shafts always need a cap-beam for structural soundness. This cap-beam may be an integral part of the superstructure.

13.4.7.3 Piers

Figures 13.4B and 13.4C provide schematics in the plan and elevation views of various pier types. Generally, the round column (Figure 13.4B(a)) is the most economical, because it is easy to construct and performs well seismically.

In debris-prone or ice-prone streams, the wall-type pier or single column with hammerhead is preferred.

Piers located in waterways susceptible to ice accumulation should be fitted with steel ice protection (Figure 13.4B(e)).

13.4.7.4 Frame Bents

Concrete frame bents are favored to support a variety of steel and concrete superstructures. The columns of the bent can be either circular or rectangular in cross section.

Figure 13.4D illustrates the most common type of concrete bent consisting of vertical columns and a cap beam, which is typically used for highway grade separation structures.

13.4.8 Foundations

13.4.8.1 Coordination with Geotechnical Section

The selection of the foundation type is a collaborative effort between the Bridge Bureau and Geotechnical Section based on the Geotechnical Report, expected superstructure type, scour potential, etc.

The Bridge Bureau typically provides the Geotechnical Section with the applicable loads and an initial recommendation for foundation type. The Geotechnical Section reviews the foundation type recommended by the Bridge Bureau and provides the Bridge Bureau with the engineering data for foundation design.

See Chapter Two "Bridge Project Development Process" of the **Montana Structures Manual** for the timing between the Geotechnical Section and Bridge Bureau for foundation design.

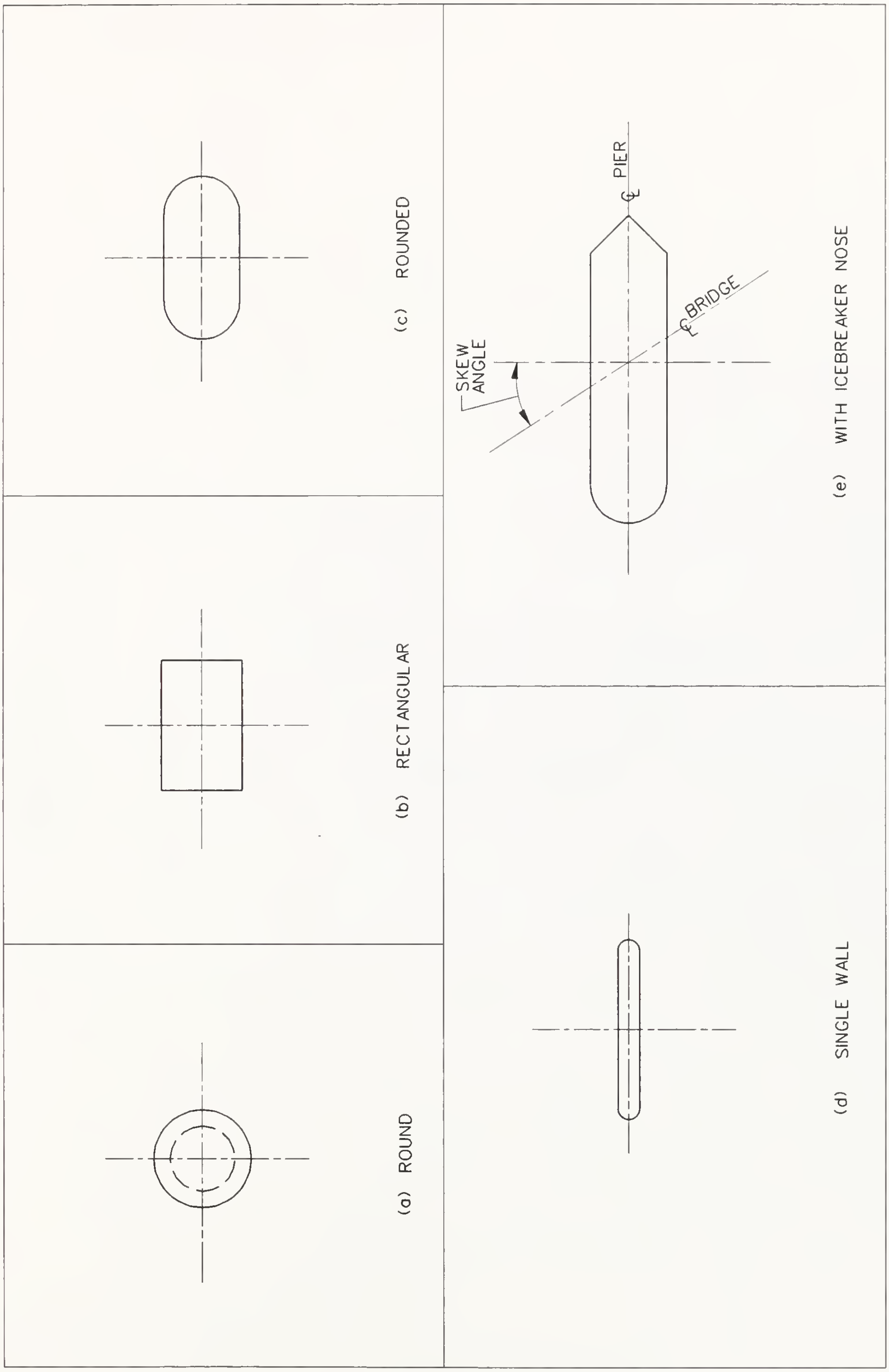
13.4.8.2 Impact on Superstructure Type

The detailed foundation study is typically performed after the superstructure selection. Therefore, the designer must anticipate the nature of the foundation characteristics in the analysis. The following should be considered:

1. Number of Supports. The expected foundation conditions will partially determine the number of and spacing of the necessary substructure supports. This will have a significant impact on the acceptable span lengths.

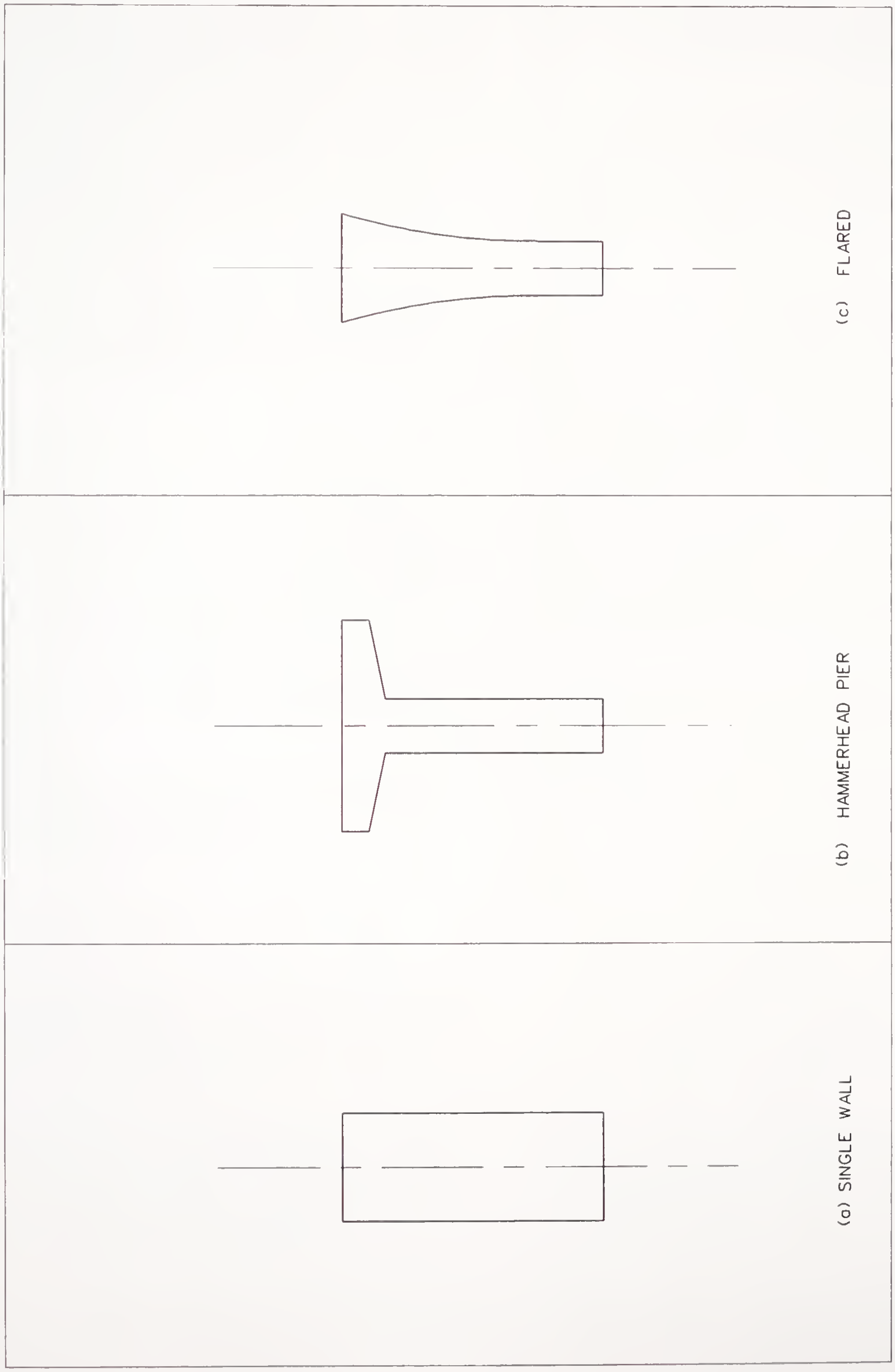
2. Dead Load. When foundation conditions are generally poor, or seismic loads are high, the economics of using structural steel over concrete should be considered.

3. Scour. The geologic or historic scour may have a significant impact on the foundation design which may, in turn, have a significant impact on the superstructure-type selection.



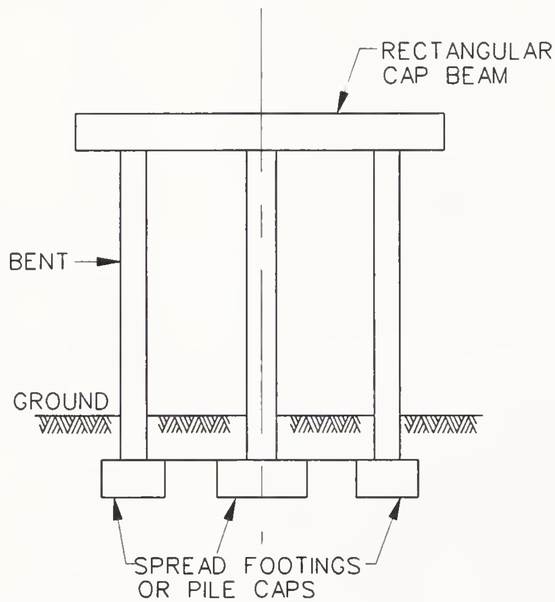
PIER STEM AND COLUMN CONFIGURATIONS
(Plan View)

Figure 13.4B



STEM CONFIGURATION OF PIERS
(Elevation View)

Figure 13.4C



CONCRETE BENT

FRAME BENTS

Figure 13.4D

13.4.8.3 Usage

The following summarizes MDT typical practices for the selection of the type of foundation:

1. Spread Footings. Use if the anticipated depth of scour is not excessive; however, do not use in fills. Spread footings are not recommended for use at stream crossings where they may be susceptible to scour. The use of spread footings requires firm bearing conditions. Spread footings normally are thick, concrete slabs whose geometry is determined by structural requirements and the characteristics of supporting components, such as soil or rock. Their primary role is to distribute the loads transmitted by piers, bents or abutments to suitable soil strata or rock at relatively shallow depths.

2. Piles. Use in soft soils or for deep bedrock; use where the anticipated depth of scour is excessive; use to control settlement. If underlying soils cannot provide adequate bearing capacity or tolerable settlements for spread footings, piles may be used to transfer loads to deeper suitable strata through friction and/or end bearing. The selected type of pile is determined by the required bearing capacity, length, soil conditions and economic considerations. MDT typically uses steel pipe piles and H-piles.

3. Drilled Shafts. Use as a deep foundation where cofferdams are a problem, where significant scour is expected, where there are limits on in-stream work or where driven piles are not economically viable due to high loads or obstructions to driving. Limitations on vibration or construction noise may also dictate the selection of this foundation type.

Figure 13.4E illustrates the basic types of foundations used by MDT. The following provides additional information with respect to interior supports at stream crossings.

13.4.8.4 Foundations For Intermediate Supports

For intermediate supports at stream crossings, MDT typically uses piles, either extended piles or piles with a pile cap footing. Where excessive scour is not expected and good load-bearing soil is close to the surface, the use of spread footings located below the anticipated depth of scour may be considered as an alternative.

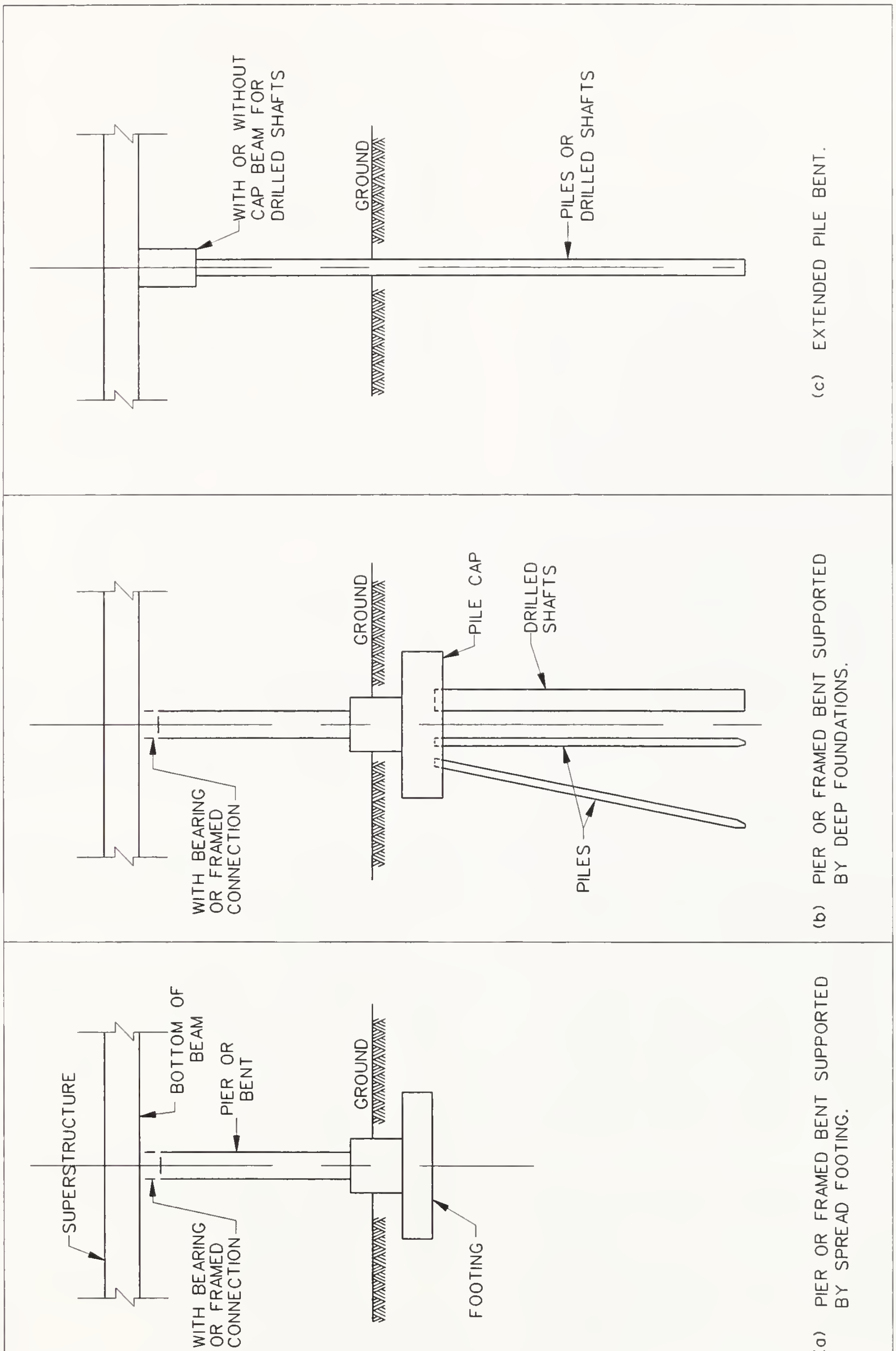
13.4.8.4.1 Supported by Spread Footing

Settlement criteria need to be consistent with the function and type of structure, anticipated service life and consequences of unanticipated movements on service performance. Angular distortions between adjacent spread-footings greater than 0.008 radians in simple spans and 0.004 radians in continuous spans should not be ordinarily permitted.

In general, spread footings require a reasonably good foundation material close to the ground surface. In all cases, the bottom of spread footings on soil should be below the frost level which, in Montana, is considered 1.5 m.

13.4.8.4.2 Supported by Deep Foundation

Where conditions that permit the use of spread footings are not present, deep foundations, such as drilled shafts or piles, should be considered.



TYPICAL FOUNDATION TYPES

Figure 13.4E

13.5 ROADWAY DESIGN ELEMENTS

The **Montana Road Design Manual** documents MDT's roadway design criteria. In general, the road design criteria will determine the proper geometric design of the roadway. The bridge design will accommodate the roadway design across any structures within the project limits. This will provide full continuity of the roadway section for the entire project. This process will, of course, require proper communication between the road designer and bridge designer to identify and resolve any problems. Section 13.5 of the **Structures Manual** provides roadway design information which is directly relevant to determining the structural dimensions for bridge design and to provide the bridge designer with some background in road design elements.

13.5.1 General Procedures

13.5.1.1 Division of Responsibilities

Determining the roadway design for a bridge or underpass is a collaborative effort between the Bridge Bureau and Road Design Section. At this stage of project development (i.e., determining the dimensions of the structure), the basic process is:

1. Preliminary Alignment. The road designer provides the Bridge Bureau with preliminary horizontal and vertical alignments.

2. Structural Dimensions. The bridge designer determines a preliminary structure length and depth of superstructure, and the bridge designer provides approximate bridge end elevations.

3. Final Alignment. The road designer modifies the alignment as necessary, based on the preliminary grade recommendations from the bridge designer. The Bridge Bureau reviews and comments on the proposed roadway geometrics.

4. Roadway Cross Section. See Section 13.5.4 for the determination of the roadway cross section (e.g., width, cross slope).

13.5.1.2 Coordination in Project Development

Chapter Two "Bridge Project Development Process" of the **Montana Structures Manual** documents the project development process for bridge projects when the Bridge Bureau is the lead unit for project development. The project networks in Chapter Two illustrate the timing of the interaction between the Road Design Section and Bridge Bureau in determining the roadway design across a bridge and through an underpass. The bridge designer should refer to Chapter Two for coordination in project development.

Chapter One "Road Design Process" of the **Montana Road Design Manual** documents the project development process for roadway projects when the Road Design Section is the lead unit for project development. Where a bridge is within the limits of a road-lead project, the bridge designer should refer to Chapter One for coordination.

13.5.1.3 Scope of Bridge Work

The determination of the roadway design for a bridge is based on the scope (or extent) of the proposed structural work. See Section 13.1.1.

Chapter Thirteen of the **Montana Structures Manual** provides roadway design criteria only for a new bridge and bridge replacement projects. Chapter Twenty-two provides roadway design criteria for all other scopes of bridge work.

13.5.2 Roadway Definitions

Chapter Twenty-one of the **Montana Road Design Manual** provides an in-depth glossary of terms which are used in road design. The following defines selected roadway elements

which often have an application to the roadway design portion of a bridge:

1. Average Annual Daily Traffic (AADT). The total yearly volume in both directions of travel divided by the number of days in a year.

2. Average Daily Traffic (ADT). The total traffic volumes in both directions of travel in a time period greater than one day and less than one year divided by the number of days in that time period.

3. Bridge Roadway Width. The clear width of the structure measured at right angles to the center of the roadway between the bottom of curbs or, if curbs are not used, between the inner faces of parapet or railing.

4. Cross Slope. The slope in the cross section view of the travel lanes, expressed as a percent, based on the change in vertical compared to the change in horizontal.

5. Design Exception. The process of receiving approval from the FHWA or MDT Preconstruction Engineer for using geometric design criteria which does not meet the criteria set forth in the **Montana Road Design Manual**.

6. Design Speed. The selected speed used to determine the various geometric design features of the roadway.

7. Grade Slope. The rate of slope between two adjacent VPIs expressed as a percent. The numerical value for percent of grade is the vertical rise or fall in meters for each 100 m of horizontal distance. Upgrades in the direction of stationing are identified as plus (+). Downgrades are identified as minus (-).

8. K-Values. For a crest or sag vertical curve, the horizontal distance (in meters) needed to produce a 1% change in longitudinal gradient.

9. Median. The portion of a divided highway separating the two traveled ways for traffic in opposite directions. The median width includes both inside shoulders.

10. Normal Crown (NC). The typical cross section on a tangent section referenced to centerline with equal downslope to the edge of pavement.

11. Profile Grade Line. A series of tangent lines connected by vertical curves. It is typically placed along the roadway centerline of undivided facilities and at the edges of the two roadways on the median side on divided facilities.

12. Roadway. The portion of a highway, including shoulders, for vehicular use. A divided highway has two roadways.

13. Superelevation. The amount of cross slope or "bank" provided on a horizontal curve to help counterbalance the outward pull of a vehicle traversing the curve.

14. Superelevation Transition Length. The distance required to transition the roadway from a normal crown section to the full superelevation. Superelevation transition length is the sum of the tangent runout and superelevation runoff distances.

15. Traveled Way. The portion of the roadway for the through movement of vehicles, exclusive of shoulders and auxiliary lanes.

13.5.3 Highway Systems

Section 8.2 of the **Montana Road Design Manual** discusses the functional classification system and the Federal-aid system. Because of their impact on the roadway design elements of a structure, the following discussion presents a summary of these two systems.

13.5.3.1 Functional Classification System

The functional classification concept is one of the most important determining factors in highway design. The system recognizes that the public highway network serves two basic and often conflicting functions — travel mobility and access to property. In the functional classification scheme, the overall objective is that the highway system, when viewed in its entirety, will yield an optimum balance between its access and mobility purposes.

The functional classification system provides the guidelines for determining the geometric design of individual highways and streets. Based on the function of the facility, the designer can select an appropriate design speed, roadway width, roadside safety elements, amenities and other design values. The **Montana Road Design Manual** is based upon this systematic concept to determining geometric design.

The Rail, Transit and Planning Division has functionally classified all public roads and streets within Montana. For road design, it is necessary to identify the predicted functional class of the road for the selected design year (e.g., 20 years beyond the project completion date). The Rail, Transit and Planning Division will provide this information to the designer.

The following briefly describes the functional characteristics of the various classifications.

13.5.3.1.1 Arterials

Arterial highways are characterized by a capacity to quickly move relatively large volumes of traffic and an often restricted function to serve abutting properties. The arterial system typically provides for high travel speeds and the longest trip movements. The arterial functional class is subdivided into principal and minor categories for rural and urban areas.

Principal arterials provide the highest traffic volumes and the greatest trip lengths. The freeway, which includes Interstate highways, is the highest level of arterial. In rural areas, minor arterials will provide a mix of interstate and interregional travel service. In urban areas, minor arterials may carry local bus routes and provide intra-community connections.

13.5.3.1.2 Collectors

Collector routes are characterized by a roughly even distribution of their access and mobility functions. Traffic volumes will typically be somewhat lower than those of arterials. In rural areas, collectors serve intra-regional needs and provide connections to the arterial system. In urban areas, collectors act as intermediate links between the arterial system and points of origin and destination.

13.5.3.1.3 Local Roads and Streets

All public roads and streets not classified as arterials or collectors are classified as local roads and streets. These facilities are characterized by their many points of direct access to adjacent properties and their relatively minor value in accommodating mobility.

13.5.3.2 Federal-Aid System

The Federal-aid system consists of those routes within Montana which are eligible for the categorical Federal highway funds. The Department, working with the local governments and in cooperation with FHWA, has designated the eligible routes. The following briefly describes the components of the Federal-aid system.

13.5.3.2.1 National Highway System

The National Highway System (NHS) is a system of those highways determined to have the greatest national importance to transportation, commerce and defense in the United States. It consists of the Interstate

highway system, logical additions to the Interstate system, selected other principal arterials, and other facilities which meet the requirements of one of the subsystems within the NHS.

13.5.3.2.2 Surface Transportation Program

The Surface Transportation Program (STP) is a block-grant program which provides Federal-aid funds for any public road not functionally classified as a minor rural collector or a local road or street. The STP replaced a portion of the former Federal-aid primary system and replaced all of the former Federal-aid secondary and urban systems, and it includes some collector routes which were not previously on any Federal-aid system. Collectively, these are called Federal-aid Roads. In addition, bridge projects using STP funds are not restricted to Federal-aid Roads but may be used on any public road. The basic objective of the STP is to provide Federal funds for improvements to facilities not considered to have significant national importance with a minimum of Federal requirements for funding eligibility.

13.5.3.2.3 Highway Bridge Replacement and Rehabilitation Program

Because of the nationwide emphasis on bridges, the Highway Bridge Replacement and Rehabilitation Program (HBRRP) has its own separate identity within the Federal-aid program. HBRRP funds are eligible for work on any bridge on a public road regardless of its functional classification. Chapter Twenty-two of the **Montana Structures Manual** briefly discusses the MDT implementation of the HBRRP within the context of bridge rehabilitation projects. The use of HBRRP funds for bridge removal is not appropriate.

13.5.4 Roadway Cross Section (Bridges)

Section 13.6 presents criteria for roadway widths across rural bridges based on the type of highway, and it presents several typical bridge sections. Section 13.5.4 presents additional information on the roadway cross sections for bridges.

13.5.4.1 Montana Road Design Manual

Chapters Eleven and Twelve of the **Montana Road Design Manual** provide the MDT criteria for the various cross section elements for the roadway. This includes lane and shoulder widths, cross slopes, auxiliary lanes, parking lanes, medians, side slopes and sidewalks. Section 11.7 of the **Road Design Manual** provides numerous typical sections for various combinations of functional classification, rural/urban location, type of median and multilane/two-lane roadways. Chapter Twelve provides detailed tables of geometric design criteria for various classes of highway and rural/urban location. Sections 13.5.4 and 13.6 of the **Montana Structures Manual** provide MDT criteria for roadway cross section elements specifically across bridges.

13.5.4.2 Profile Grade Line

The location of the profile grade line on the bridge must match the location of the profile grade line on the approaching roadway. The profile grade line location varies according to the functional class of the highway, divided versus undivided, type of median and median width. See Section 10.2.3.2 of the **Montana Road Design Manual** for MDT criteria.

13.5.4.3 Cross Slopes and Crowns

All bridges on tangent sections provide a uniform cross slope of 2% from the crown line to the edge of the bridge curb, parapet or rail. If the traveled way crown slope on the approaching roadway is other than 2%, the roadway must be transitioned to a uniform 2% slope before it reaches the bridge; this is the responsibility of the road designer when designing the roadway approaches. If the roadway section is

superelevated, the bridge cross slope will match the superelevation rate of the roadway.

13.5.4.4 Width

The Bridge Bureau determines the bridge width according to the criteria presented in Sections 13.5.4 and 13.6. However, this width must not be less than the roadway width presented in Chapter Twelve of the **Montana Road Design Manual**.

The bridge width will be determined by:

1. the functional class of the highway;
2. the approaching roadway width;
3. the presence of sidewalks and/or bikeways;
4. the presence of auxiliary lanes;
5. for divided facilities, whether a single or dual structure is used; and
6. AADT and DHV.

See Section 13.6 for specific bridge width criteria.

13.5.4.5 Sidewalks

13.5.4.5.1 Guidelines

Sidewalk requirements on bridges will be determined jointly by the Bridge Bureau, the Road Design Section and the District. On bridge rehabilitation projects, the need for sidewalks will be considered on a case-by-case basis. The following guidance will be used to determine the need for sidewalks on a bridge:

1. Sidewalks Currently Exist. If a bridge with an existing sidewalk is replaced or reconstructed, the sidewalk will normally be replaced.
2. Bridge Without Sidewalk/Roadway With Sidewalk. If a bridge without a sidewalk will be replaced and if existing sidewalks approach the bridge, a sidewalk will normally be included in the bridge project. Even if not currently on the approaching roadway, sidewalks may still be necessary on the bridge if the approach roadway is a candidate for future sidewalks.

   As a more general statement of MDT policy, bridge projects within urban areas will have a sidewalk where pedestrians are legally allowed, unless there is a compelling reason not to provide a sidewalk. In addition, bridges at interchanges near urban areas should normally include sidewalks to accommodate the commercial development that typically occurs in the immediate vicinity of interchanges.
3. One Side vs. Two Sides. Sidewalk requirements for each side of the bridge will be evaluated individually; i.e., placing a sidewalk on each side will be based on the specific characteristics of that side.
4. Approval. For all projects in urban areas, the final decision on sidewalk requirements will be made by the Preconstruction Engineer.

13.5.4.5.2 Cross Section

The typical sidewalk configuration is shown in Figure 13.6D.

13.5.4.6 Bikeways

The bicycle is classified as a vehicle according to the **Montana Codes Annotated**, and bicyclists are granted all of the rights and are subject to all of the duties applicable to the driver of any other vehicle. Section 18.2 of the **Montana Road Design Manual** provides definitions for types of bikeways (i.e., bicycle path, bicycle lane and widened shoulders) and provides criteria for bikeway design.

A bridge may need to be configured to accommodate bicycle traffic. One possible accommodation is to provide wider traffic lanes (i.e., a 4.2-m minimum width). The preferred accommodation is to provide a shoulder wide enough to accommodate bicycles. Although a 1.2-m wide shoulder is considered adequate for bicycle traffic, this needs to be increased by 300 mm to provide a shy distance where curbs or barriers are present. Therefore, a 1.5-m wide shoulder is considered the minimum shoulder width for bridges that are anticipated to carry bicycle traffic.

If the approaching roadway includes a separate bicycle lane, then the width of the lane will be carried across the bridge. Requests for and consideration of anticipated future bicycle lanes are only warranted when they are part of the Master Transportation Plan for the local government. Bicycle issues must be discussed at the Preliminary Field Review to establish a consensus on the level of bicycle accommodation that may be required on specific projects.

13.5.4.7 Medians

For divided facilities, the bridge designer must decide if one structure will be used for the entire roadway section (including the median) or if dual structures will be used. This will be determined on a case-by-case basis. As a general rule, a single structure will be used for roadways with a flush or raised median, and dual structures will be used for roadways with a depressed median. See Section 11.3 of the **Montana Road Design Manual** for more information on medians.

13.5.5 Alignment at Bridges

13.5.5.1 Horizontal Alignment

The road designer will determine the horizontal alignment at the bridge based on Chapter Nine of the **Montana Road Design Manual** (e.g., curve radius, superelevation transition). From the perspective of the roadway user, a bridge is an integral part of the roadway system and, ideally, horizontal curves and their transitions will be located irrespective of their impact on bridges. However, practical factors in bridge design and bridge construction warrant consideration in the location of horizontal curves at bridges. The following presents, in order from the most desirable to the least desirable, the application of horizontal curves to bridges:

1. From both the complexity of design and construction difficulty, the most desirable treatment is to locate the bridge and its approach slabs on a tangent section; i.e., no portion of the curve or its superelevation development will be on the bridge or bridge approach slabs.

2. If a horizontal curve is located on a bridge, any transitions should not be located on the bridge or its approach slabs. This includes both superelevation transitions and spiral transitions. This will result in a uniform cross slope (i.e., the design superelevation rate) and a constant rate of curvature throughout the length of the bridge and bridge approach slabs.

3. If the superelevation transition is located on the bridge or its approach slabs, the designer should place on the roadway approach that portion of the superelevation development which transitions the roadway cross section from its normal crown to a point where the roadway slopes uniformly; i.e., to a point where the crown has been removed. This will avoid the need to warp the crown on the bridge or the bridge approach slabs.

13.5.5.2 Vertical Alignment

As discussed in Section 13.5.1, the road designer and bridge designer will collaborate on the vertical alignment or profile of the roadway across a bridge. Chapter Ten of the **Montana Road Design Manual** provides the Department's criteria. The following applies specifically to the vertical alignment at bridges:

1. Minimum Gradient. For bridges with a parapet rail, the minimum longitudinal gradient is 0.2%. For bridges with an open rail, the minimum longitudinal gradient is 0.0%.

2. Maximum Grades. See Chapter Twelve of the **Montana Road Design Manual** for the Department's maximum grade criteria based on the highway system and rural/urban location.

3. Vertical Curves. Vertical curves will be designed according to Chapter Ten of the **Montana Road Design Manual**. Where bridges are located on sag or crest vertical curves, consideration must be provided to haunch depths and shear connector heights.

13.5.5.3 Skew

The maximum skew angle on a bridge without approval is 35°. The Bridge Area Engineer must approve the use of greater skew angles. Also, the bridge skew should not match the angle of a snowplow, which is 35° to 37° right.

13.5.6 Roadway Cross Section (Underpasses)

For highway bridges over highways, the design of the underpassing roadway will determine the length of the overpassing bridge. Section 13.6 presents several typical sections for bridge underpasses. Section 13.5.6 presents additional information on roadway design elements for underpasses.

13.5.6.1 Roadway Section

The approaching roadway cross section, including any auxiliary lanes, should be carried through the underpass. Chapters Eleven and Twelve of the **Montana Road Design Manual** present the MDT criteria for roadway widths based on highway system and rural/urban location.

13.5.6.2 Roadside Clear Zones

Desirably, the roadside clear zone applicable to the approaching roadway section will be provided through the underpass. Chapter Fourteen of the **Montana Road Design Manual** provides MDT criteria for clear zones, which are based on design speed, traffic volumes and side slope configuration. If this is impractical, an alternative is to provide roadside barrier protection for piers, walls, abutments, etc., located within the clear zone.

13.5.6.3 Sidewalks/Bikeways

The principles and design criteria expressed in Section 13.5.4.5 and 13.5.4.6 for sidewalks and bikeways on bridges also apply to underpasses.

13.5.6.4 Vertical Clearances

The vertical clearance for underpassing roadways will significantly impact the vertical location of the overpassing structure and may dictate the selection of the superstructure type. Chapter Twelve of the **Montana Road Design Manual** presents the Department's vertical clearance criteria for underpassing roadways based on type of highway and rural/urban location.

13.5.6.5 Future Expansion

When determining the cross section width of an underpass, the designer should also consider the likelihood of future roadway widening. Widening an existing underpass can be extremely expensive, and it may be warranted, if some flexibility is available, to allow for possible future roadway expansion in the initial bridge construction. Therefore, the designer should evaluate the potential for further development in the vicinity of the underpass which would significantly increase traffic volumes. If appropriate, a reasonable allowance for future widening may be to provide sufficient

lateral clearance for one additional lane in each direction.

13.6 STRUCTURE DIMENSIONS (Design Aids)

Section 13.6 presents the following design aids to assist in determining the dimensions of the structure for preliminary design:

1. design criteria for widths on rural bridges,
2. typical sections for bridges and underpasses,
3. calculating structure lengths for stream crossings and highway crossings, and
4. end bent dimensioning and configuration.

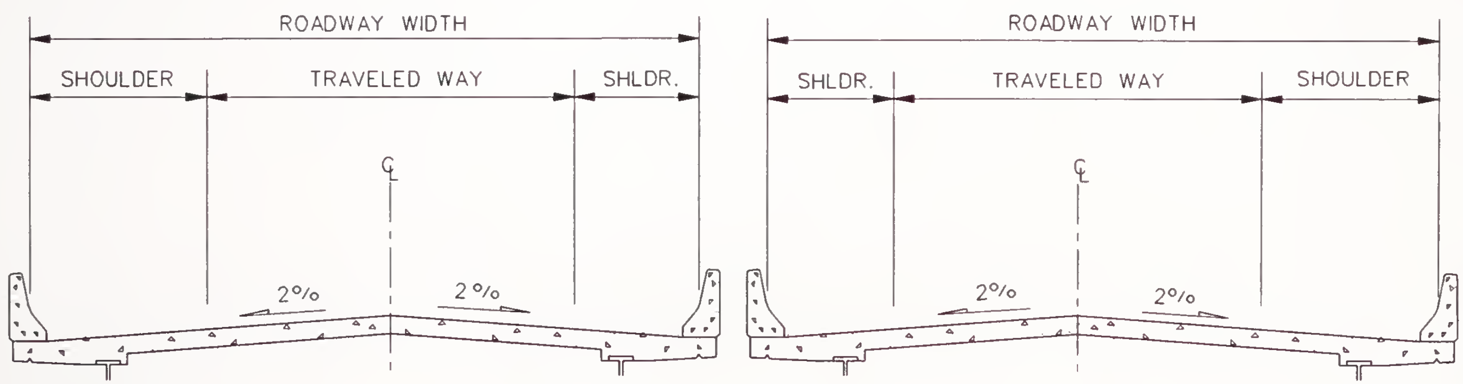
13.6.1 Rural Bridge Widths

MDT has produced the **Montana Bridge Design Standards**, which is a separate document. These Standards are among very few formally approved "Standards" currently in use by the Department. There is not total agreement among the bridge widths shown on these standards and the roadway widths in Chapter Twelve of the **Montana Road Design Manual**. Within some broad parameters, virtually all information in the **Montana Road Design Manual** is considered guidance. When deviating from the **Montana Bridge Design Standards**, formal design exception approval from the Bridge Engineer must be secured.

13.6.2 Typical Sections

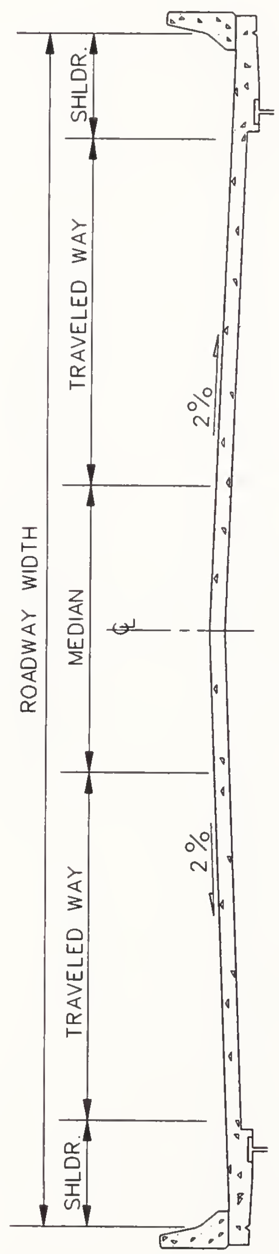
Section 13.6.2 presents typical section figures for bridges and underpasses as follows:

1. Figure 13.6A "Bridge Cross Section (Freeways/Interstates)."
2. Figure 13.6B "Bridge Cross Section (Multi-lane Highways — Single Structure)."
3. Figure 13.6C "Bridge Cross Section (Rural, Two-Lane, Two-Way Highway)."
4. Figure 13.6D "Bridge Cross Section (Urban, Two-Lane, Two-Way Highway)."
5. Figure 13.6E "Freeway Underpass Cross Section."
6. Figure 13.6F "Non-Freeway, Two-Lane Underpass Cross Section."

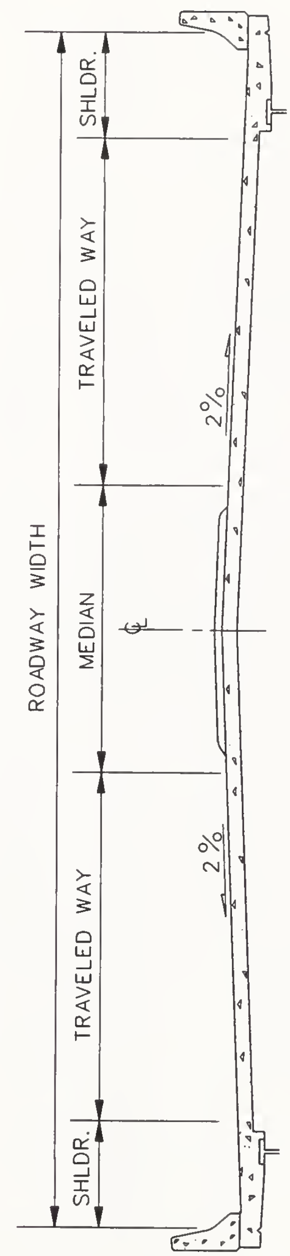


BRIDGE CROSS SECTION
(Freeways/Interstates)

Figure 13.6A



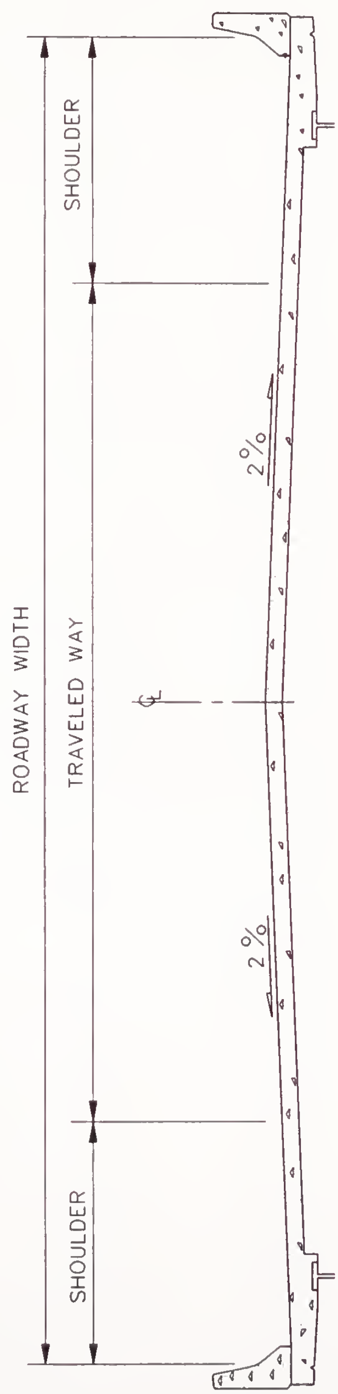
FLUSH MEDIAN



RAISED MEDIAN

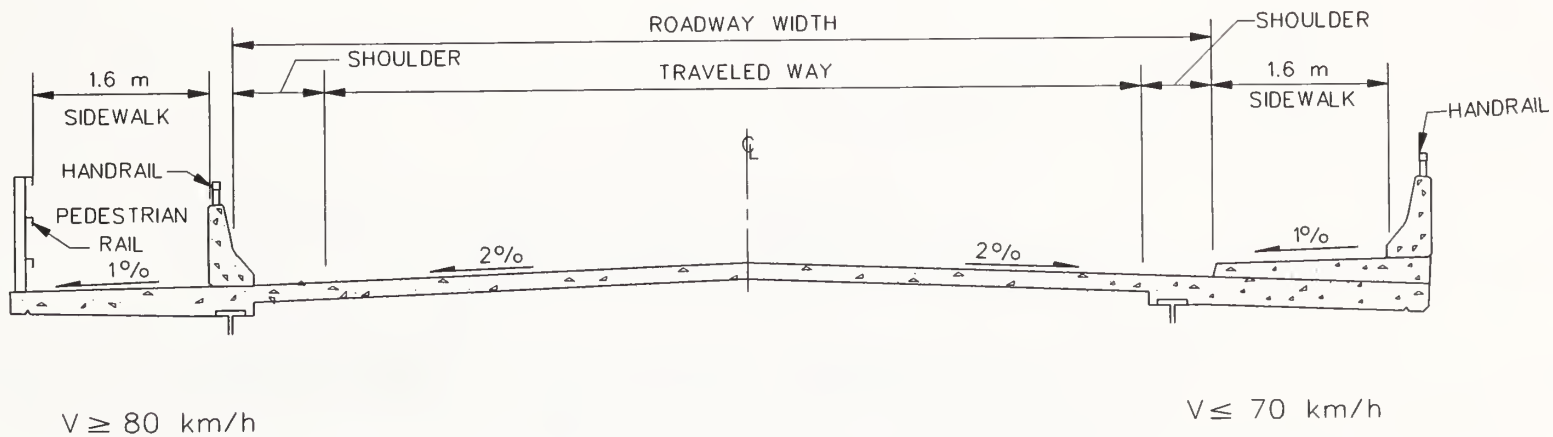
BRIDGE CROSS SECTION
(Multi-Lane Highways — Single Structure)

Figure 13.6B



BRIDGE CROSS SECTION
(Rural, Two-Lane, Two-Way Highway)

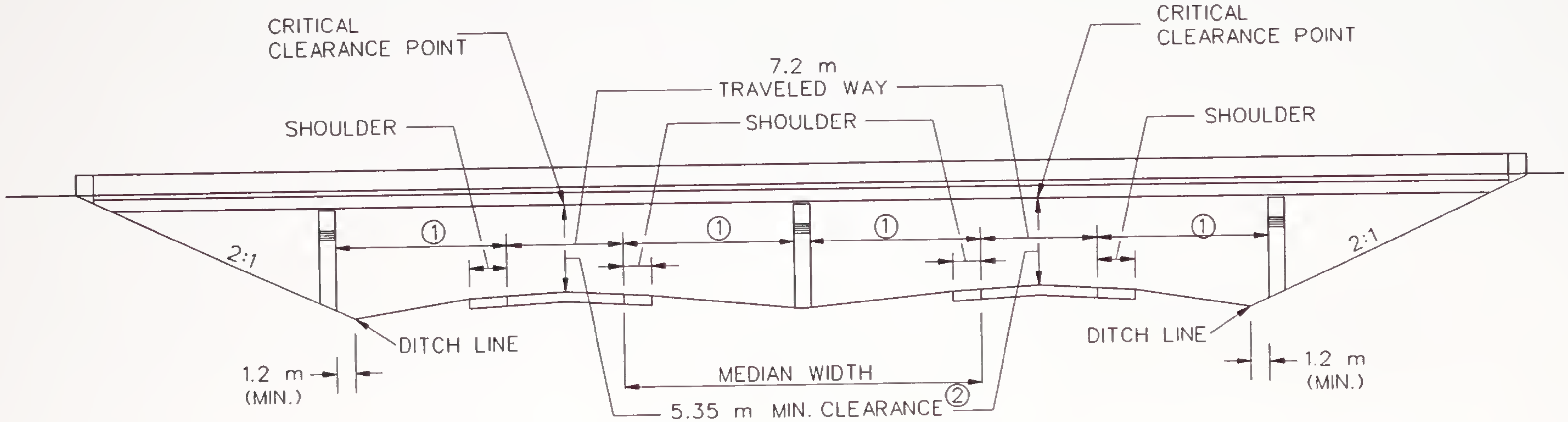
Figure 13.6C



NOTE: USE RAISED SIDEWALK ONLY IF ADEQUATE SHOULDER WIDTH IS PROVIDED. CONSIDER AADT AND DESIGN SPEED WHEN DETERMINING ADEQUATE SHOULDER WIDTH.

BRIDGE CROSS SECTION
(Urban, Two-Lane, Two-Way Highway)

Figure 13.6D

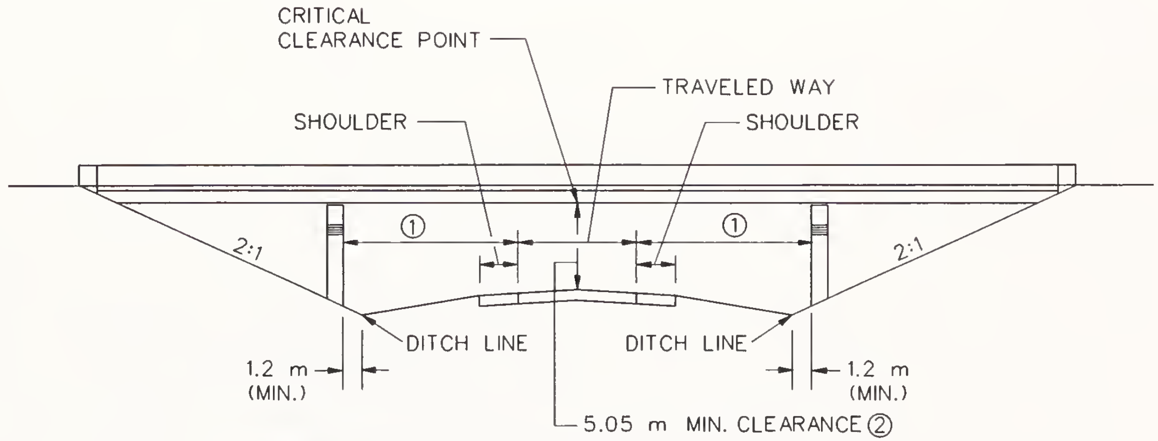


① CLEAR ZONE (SEE CHAPTER 14 MDT ROAD DESIGN MANUAL). IF CLEAR ZONE IS NOT AVAILABLE OR FEASIBLE, USE ROADSIDE BARRIER IN ACCORDANCE WITH "MDT STANDARD DRAWINGS".

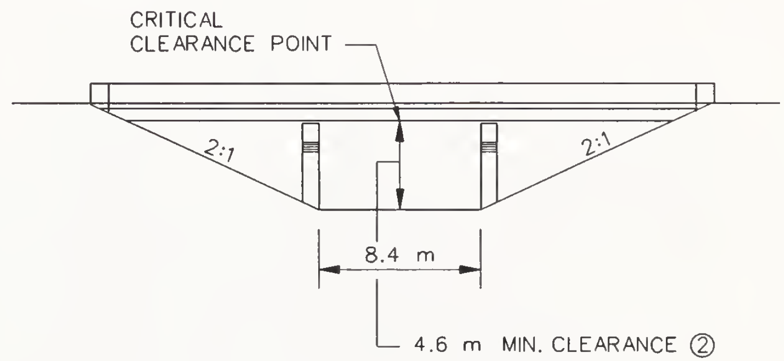
② MINIMUM ALLOWABLE CLEARANCE IS 4.9 m BUT PROVIDE 5.35 m FOR INITIAL DESIGN TO ALLOW FOR FUTURE OVERLAYS AND SNOWPACK. WITH THE HIGHER CLEARANCE, UP TO 150 mm OF SURFACING MAY BE PLACED WITHOUT THE NEED TO REVISE ROADWAY GRADES. CLEARANCES LESS THAN 4.9 m WILL BE POSTED IN ACCORDANCE WITH THE VERTICAL CLEARANCE POSTING POLICY.

FREEWAY UNDERPASS CROSS SECTION
(New Bridges/Bridge Replacements)

Figure 13.6E



ALL TWO-LANE FACILITIES
(EXCEPT LOW VOLUME, LOW SPEED ROADS)



LOW VOLUME, LOW SPEED ROADS

① CLEAR ZONE (SEE CHAPTER 14 MDT ROAD DESIGN MANUAL). IF CLEAR ZONE IS NOT AVAILABLE OR FEASIBLE, USE ROADSIDE BARRIER IN ACCORDANCE WITH "MDT STANDARD DRAWINGS".

② UP TO 150 mm OF SURFACING MAY BE PLACED WITHOUT THE NEED TO REVISE ROADWAY GRADES. CLEARANCES LESS THAN 4.9 m WILL BE POSTED IN ACCORDANCE WITH THE VERTICAL CLEARANCE POSTING POLICY.

NON-FREEWAY, TWO-LANE UNDERPASS CROSS SECTION
(New Bridges/Bridge Replacements)

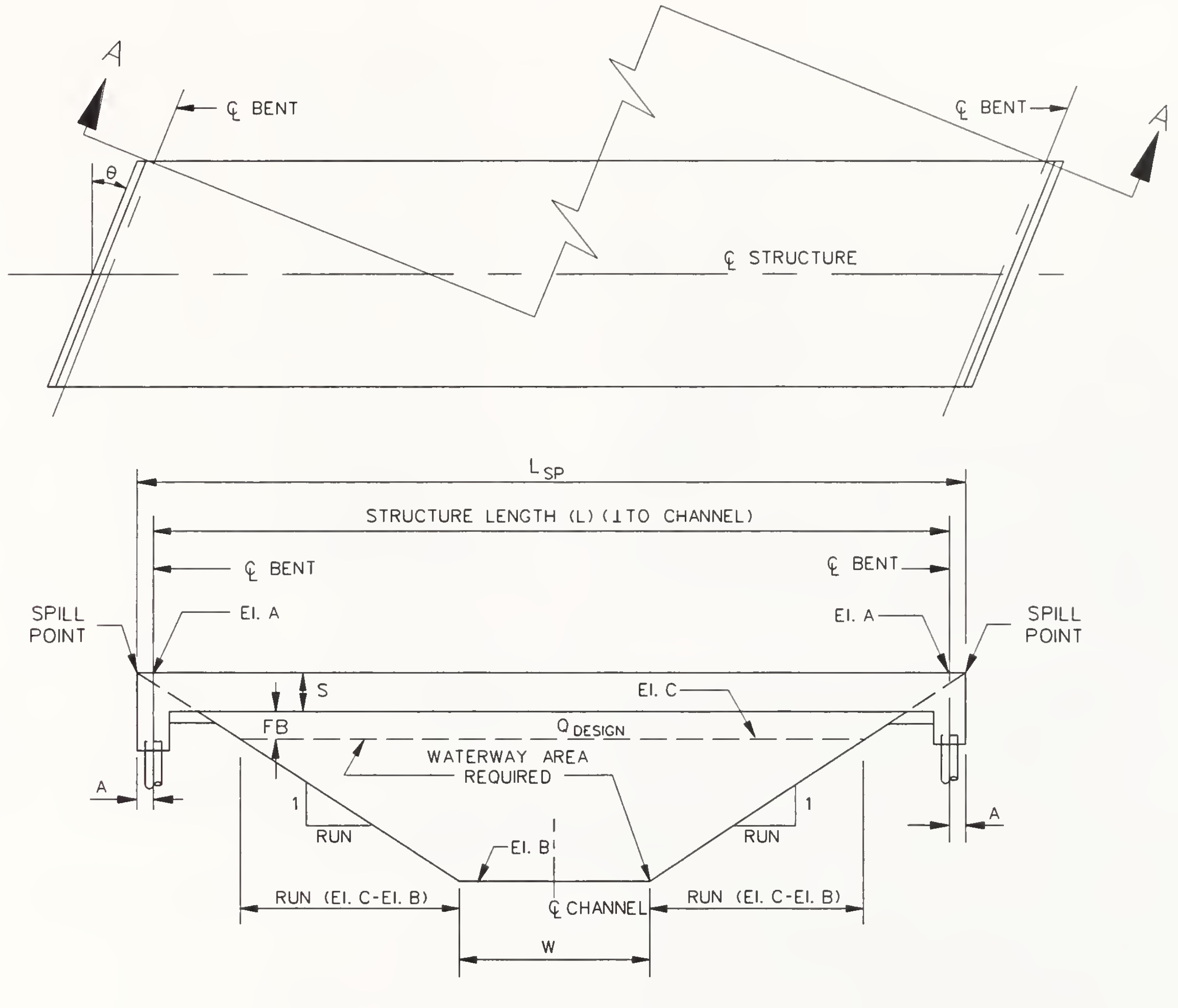
Figure 13.6F

13.6.3 Structure Length

Among other factors, structure length is determined by considering local topography, hydraulic recommendations and road design recommendations. See Figure 13.6G for an example reinforced concrete slab with an integral abutment.

To determine the approximate locations and elevations of abutments for stream crossings, use Figure 13.6G and obtain the hydraulic and road design recommendations. Use the channel bottom width, channel slopes, skew and base flood stage elevation with backwater to determine the basic bridge length.

Structure lengths for bridges over highways are determined similarly to those over stream crossings. See Chapter Twenty-one for structure lengths for highway bridges over railroads.



SECTION A-A
(PERPENDICULAR TO CHANNEL)

θ = Skew (right skew shown)
A = Spill point to centerline of bearing
S = Anticipated thickness of superstructure
FB = Distance from flood stage to bottom of superstructure
W = Width of channel (perpendicular to channel)
El. A = Elevation of top of slab
El. B = Bottom of channel elevation
El. C = Elevation of Water at Q_{DES}

Note: Minimum Waterway Area Required will be furnished by the waterway opening analysis.

Notes:

1. *Estimate minimum finished roadway grade elevation (FE):*

   FE = El.C + backwater + minimum freeboard (0.3 m) + proposed superstructure depth

2. *Structure length (L_{SP}) (along centerline roadway, spill point to spill point):*

   L_{SP} = *[(FE – channel bottom elevation) x 2 sides x run of slope + channel bottom width]/cos θ*

3. *Structure length (L) (along centerline roadway from centerline bent to centerline bent):*

   $L = L_{SP} - 2A$

STRUCTURE LENGTH FOR STREAM CROSSINGS
(Integral Abutment, Reinforced Concrete Slab)

Figure 13.6G

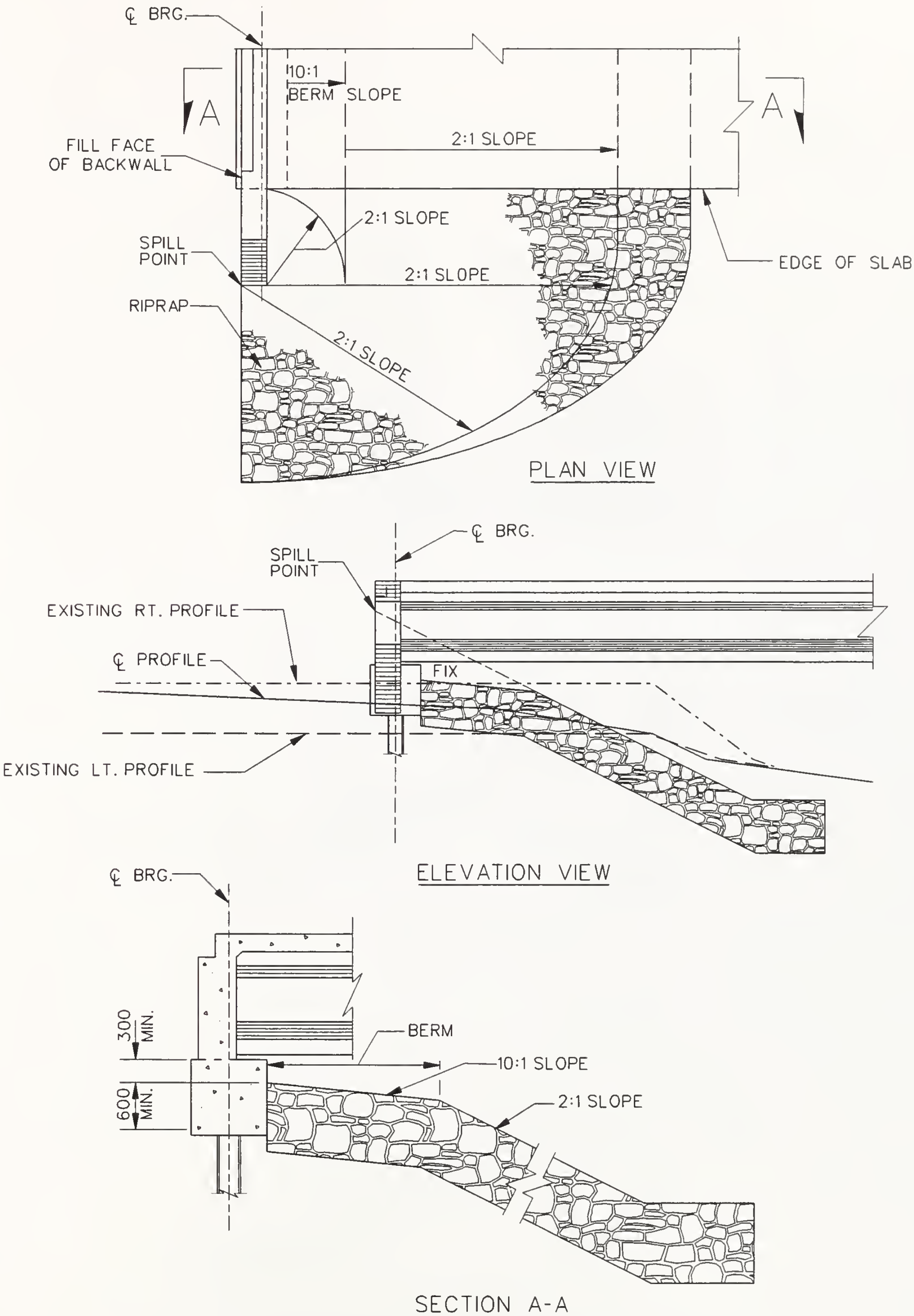
13.6.4 End Bents

End bent configuration and dimensioning has a significant impact on the required bridge span length. MDT end bents for either slab or girder bridges are typically constructed with either turn back or standard wing configurations. Turn back wings are located at and parallel to the roadway shoulders. Standard wings are constructed perpendicular to the centerline of roadway or, for a skewed bridge, parallel to the centerline of bearing. Either wing configuration is acceptable. Depending on specific site criteria and the proposed structure type, one may have advantages over the other. This must be thoroughly evaluated by the designer during the bridge layout process. Figures 13.6H, 13.6I, 13.6J and 13.6K illustrate the basic application of standard and turn back wings:

1. Berms. When standard wings are used, the end bent will typically require a berm. The berm is located in front of the cap and is constructed on a 10:1 slope to drain away from the front face of the cap. The berm length is dependent on the superstructure depth, cap width and relative difference between the berm slope and fill slope. The berm length will be determined by calculating the mathematical point of intersection of the berm slope and fill slope. The berm slope and cap slope will typically be located 300 mm below the low beam seat on the cap. Bridges with turn back wings will typically not require a berm.

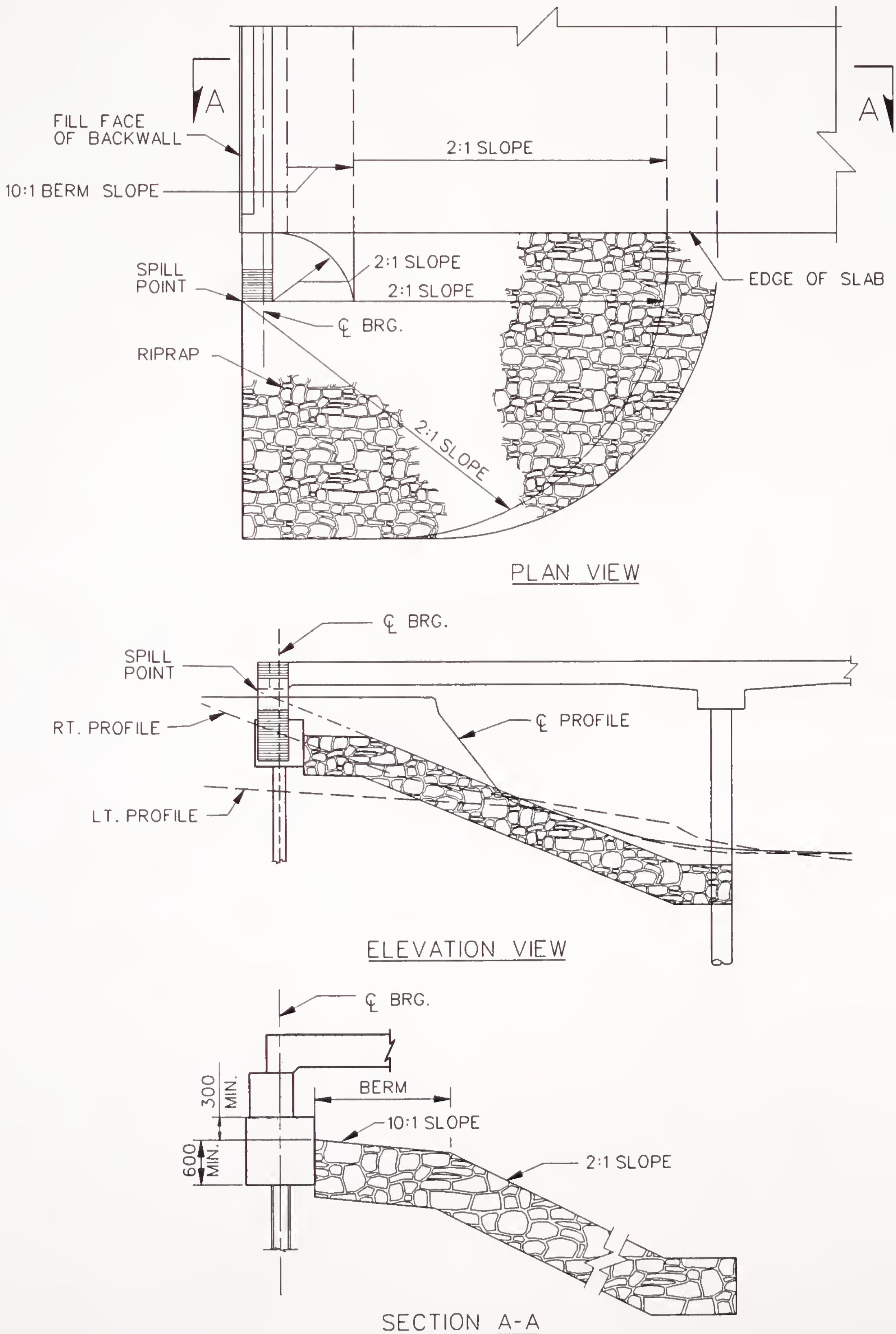
2. Slope. The fill slope is typically 2:1. This will typically be specified in the Hydraulics Report for stream crossings or by the Geotechnical Section elsewhere. In some cases, fill slopes can be increased to 1.5:1 with approval from the Hydraulics or Geotechnical Section. Slopes steeper than 1.5:1 will be engineered slopes.

3. Wingwalls. Wingwall type and length will be determined for each specific structure by the designer. Standard wing lengths are a direct function of slope and superstructure depth and are determined mathematically by the distance required for a slope running along both sides and the end of the wingwall to catch the end of the cap 300 mm below the low beam seat. The slope along the fill face of the wing out to the spill point will match the roadway prism at the bridge end including provisions for guardrail widening. The slope from the spill point across the end and near the face of the wing will be the fill slope. Turned back wing lengths can be established as needed to achieve the required bridge length. The minimum length of a turned back wing is that required for the fill slope to intersect the front face of the cap 300 mm below the low beam seat; i.e., the distance from the front of the cap to the end of the wing will typically be twice the vertical distance between the spill point at the end of wing and the fill slope intersection at the front face of the cap. If additional bridge length is needed, the walls can be lengthened but, as the wall length increases, the required cap depth will also increase. Walls exceeding 2 m in length may require piles or added support at the end of the wing.



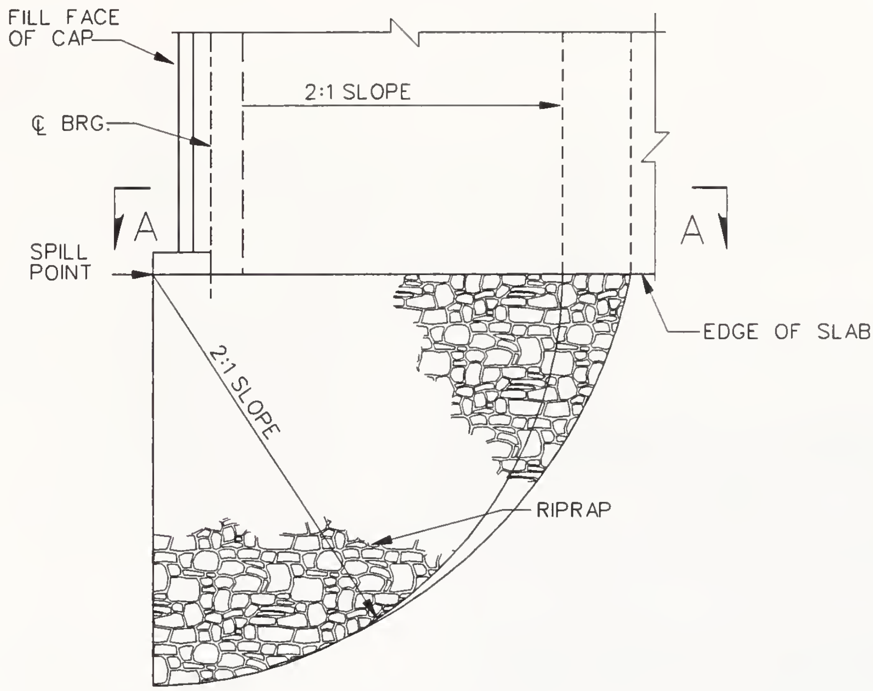
GIRDER BRIDGE WITH STANDARD WINGS

Figure 13.6H

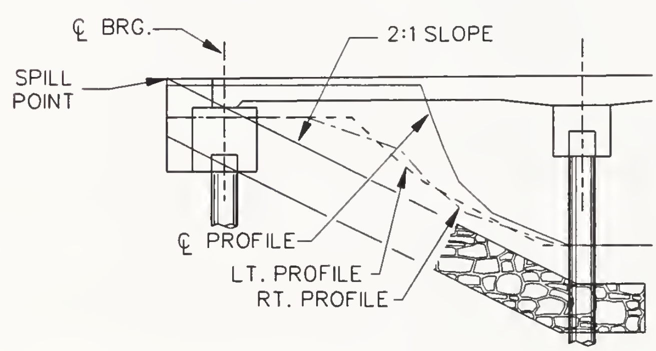


FLAT SLAB WITH STANDARD WINGS

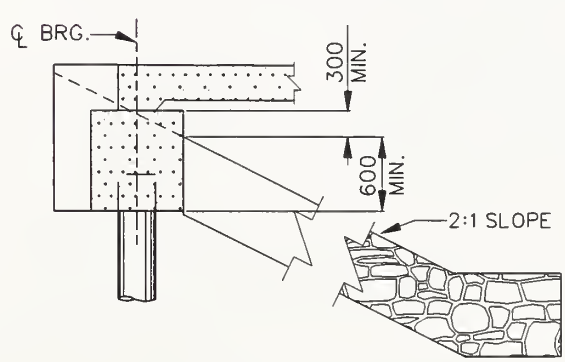
Figure 13.6I



PLAN VIEW



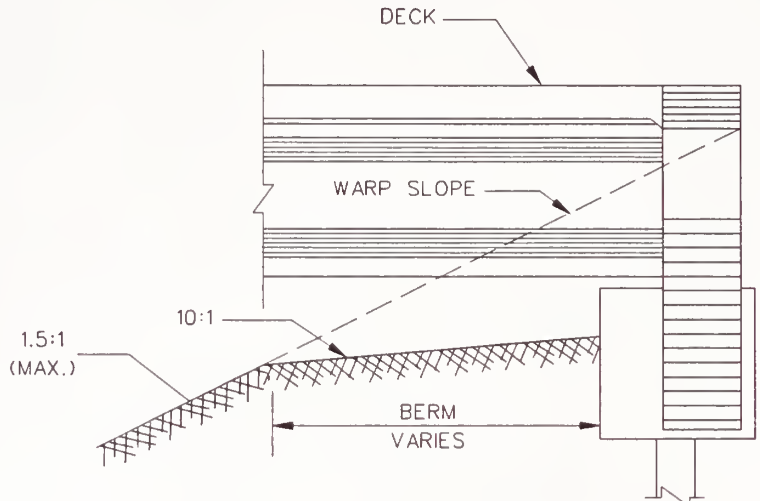
ELEVATION VIEW

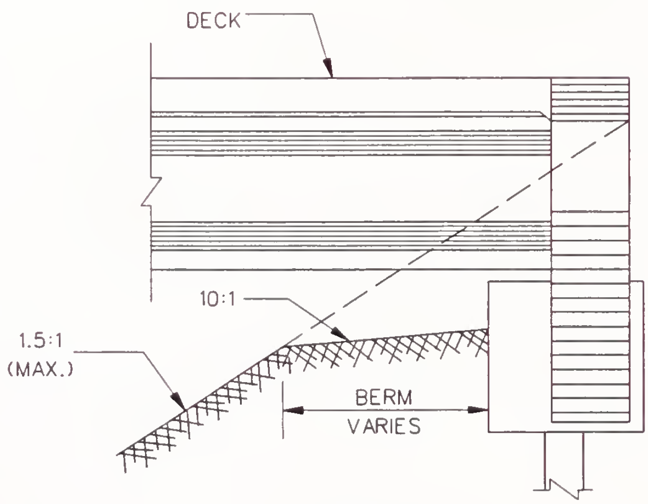


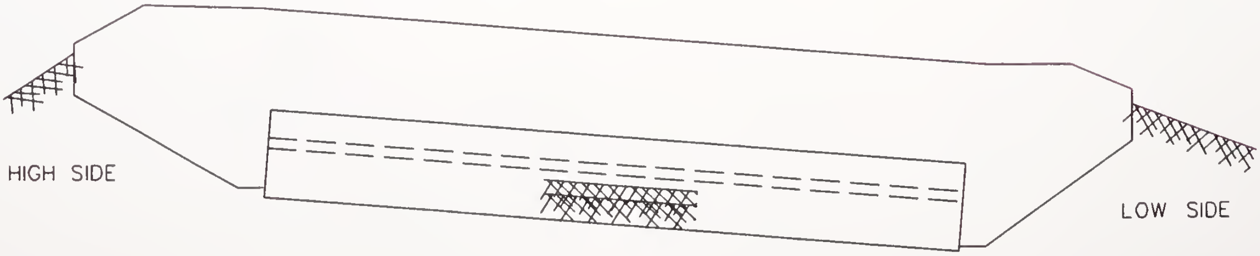
SECTION A-A

FLAT SLAB WITH TURN BACK WINGS

Figure 13.6J







BERM TREATMENT ON SUPERELEVATED CURVES

Figure 13.6K

13.7 HYDRAULICS

The **Montana Hydraulics Manual** documents MDT's hydraulic design criteria for roadways and bridge waterway openings. In general, a bridge over a waterway must be dimensioned to meet the applicable hydraulic criteria, which is a blend of Federal and State requirements (e.g., environmental, floodplains) and Department practices. This process will require proper communication between the Hydraulics Section and bridge designer to identify and resolve any problems. Section 13.7 of the **Structures Manual** provides basic hydraulic design criteria which applies to bridge waterway openings to provide the bridge designer with some background in hydraulic elements.

13.7.1 General Procedures

13.7.1.1 Division of Responsibilities

The Hydraulics Section is responsible for hydrologic and hydraulic analyses for both roadway drainage appurtenances and bridge waterway openings. The Hydraulics Section will perform the following for the design of bridge waterway openings for new bridges:

1. the hydrologic analysis to calculate the design flow rates based on the drainage basin characteristics;
2. the hydraulic analysis to determine the necessary dimensions of the bridge waterway opening to pass the design flood, to meet the backwater allowances and to satisfy any regulatory floodplain requirements; and
3. the hydraulic scour analysis to assist in determining the proper foundation design for the new bridge.

Based on the hydraulic analysis, the Hydraulics Section will provide the following to the Bridge Bureau for new bridges:

1. the water surface elevation for the design-year flood,
2. a suggested low beam elevation,
3. the necessary bridge waterway opening dimensions, skew angle and channel centerline station, and
4. the results of its hydraulic scour analysis.

The Hydraulics Section is also responsible for determining that the bridge design is consistent with regulations promulgated by the Federal Emergency Management Agency (e.g., development within regulatory floodplains).

The Hydraulics Section will submit the necessary information to the Bridge Bureau documenting its recommendations for the hydraulic design of the bridge typically via a memorandum. The bridge designer will incorporate these details into the bridge design.

13.7.1.2 Coordination in Project Development

Chapter Two "Bridge Project Development Process" of the **Montana Structures Manual** documents the project development process for bridge projects. The project networks in Chapter Two illustrate the timing of the interaction between the Hydraulics Section and Bridge Bureau for waterway openings. For a new bridge/bridge replacement project, the Hydraulics Section will perform its hydraulic analysis before the Bridge Bureau prepares the preliminary bridge layout and establishes bridge end elevations.

13.7.2 Hydraulic Definitions

The following presents selected hydraulic definitions which have an application to bridge design:

1. Auxiliary Waterway Openings. Relief openings provided for streams in floodplains through the roadway embankment in addition to the primary bridge waterway opening.

2. Backwater. The incremental increase in water surface elevation upstream of a highway facility.

3. Base Flood. The flood having a 1% change of being exceeded in any given year.

4. Base Floodplain. The area subject to flooding by the base flood.

5. Bridge Waterway Opening. The opening provided in the roadway embankment intended to pass the stream flow under the design conditions.

6. Design Flood Frequency. The flood frequency selected for determining the necessary size of the bridge waterway opening.

7. Flood Frequency. The number of times a flood of a given magnitude can be expected to occur on average over a long period of time.

8. Freeboard. The clearance between the water surface elevation based on the design flood and the low chord of the superstructure.

9. Hydrology. The science which explores the interrelationship between water on the earth and in the atmosphere. In hydraulic practice for highways, hydrology is used to calculate discharges for a given site based on the site characteristics. Hydraulic methodologies include:

   a. Rational Method,

   b. USGS Regression Equations,

   c. NRCS (formerly SCS) Unit Hydrograph, and

   d. HEC I.

10. Maximum Allowable Backwater. The maximum amount of backwater which is acceptable to the Department for a proposed facility based on State and Federal laws and on Department policies.

11. Maximum Allowable Velocity. The maximum acceptable velocity through the waterway opening during the design flood.

12. 100-Year Flood Frequency. A flood volume (or discharge) level which has a 1% chance of being equaled or exceeded in any given year.

13. Overtopping Flood. That flood event which produces a discharge which will overtop the elevation of the bridge.

14. Peak Discharge (or Peak Flow). The maximum rate of water flow passing a given point during or after a rainfall event or snow melt. The peak discharge for a 100-year flood is expressed as Q_{100}.

15. Recurrence Interval (Return Period). The average number of years between occurrences of a discharge that equals or exceeds that discharge. For example, the recurrence interval for a 100-year flood discharge is 100 years.

16. Regulated Floodway. The floodplain area that is reserved in an open manner by Federal, State or local requirements (i.e., unconfined or unobstructed either horizontally or vertically) to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount as established by the Federal Emergency Management Agency (FEMA) for administering the National Flood Insurance Program (NFIP).

17. Review Flood Frequency. A frequency other than the design frequency used to assess flood hazards for the proposed

structure as part of the evaluation of the bridge waterway opening and foundation design.

18. River Stage. The water surface elevation above some elevation datum.

19. Scour. The action at a bridge foundation in which the movement of the water erodes the channel soil which surrounds the foundation. There are several types of scour:

    a. Contraction. A constriction of the channel (i.e., the flow area) which may be caused, for example, by bridge piers.

    b. Local. Removal of material from around piers, abutments, embankments, etc., due to high local velocities or flow disturbances such as eddies and vortices.

    c. Natural. Long-term aggradation and degradation of the stream bed due to natural phenomena.

20. Stream Morphology. The form and shape of the stream path created by its erosion and deposition characteristics. Streams are generally considered one of the following types:

    a. Braided. One consisting of multiple and interlacing channels.

    b. Straight. One in which the ratio of the length of the path of deepest flow to the length of the valley proper is less than 1.5. This ratio is called the sinuosity of the stream.

    c. Meandering. One consisting of alternating bends of an S-shape.

21. Thalweg. The path of deepest flow.

22. Water Surface Profile (WSPRO) Analysis. The hydraulic analysis model typically used by MDT for analyzing waterway openings. WSPRO is an "energy" model which models pressure flow through the bridge, embankment overtopping and flow through multiple openings.

13.7.3 Hydraulic Design Criteria

The following presents MDT's hydraulic criteria used for the design of bridge waterway openings:

1. Design Flood Frequency. Typically, the 100-year flood frequency is used for design.

2. Review Flood Frequency. In some cases, the impacts of the 500-year flood (or super flood) on the surrounding area may be evaluated as part of the analysis.

3. Maximum Allowable Backwater. On delineated floodplains, no backwater may be introduced by the structure. For all other sites, the maximum allowable backwater shall be limited to an amount which will not result in damage to upstream property or to the highway. The Hydraulics Section will determine the allowable backwater for each site.

4. Freeboard. Where practical, a minimum clearance of 300 mm should be provided between the design water surface elevation and the low chord of the bridge to allow for passage of ice and debris. Where this is not practical, the clearance should be established by the designer based on the type of stream and level of protection desired. For example, 150 mm may be adequate on small streams that normally do not transport drift. Urban bridges with grade limitations may not provide any freeboard. A freeboard greater than 300 mm is desirable on major rivers which may carry large debris. On bridge replacement projects, efforts should be made to match pre-existing low beam elevations. For new crossings on waterways that have substantial use by recreational boaters, consideration may need to be given to provide adequate freeboard for floaters and fishermen to pass at flows not more than Q_2.

5. Substructure Displacement. When the Hydraulics Section provides required waterway openings, an allowance has already been made for the area displaced by the substructure. Therefore, the area of piers and bents below the Q_{100} elevation should not be deducted from the gross waterway area given.

13.7.4 Good Hydraulic Practices

Before the bridge designer determines the type of structure, identifies its location and dimensions, and prepares the preliminary bridge layout and grade, he/she will have received the Hydraulics Report from the Hydraulics Section. At a minimum, the bridge waterway opening must conform to the basic hydraulic design criteria presented in Section 13.7.3.

Hydraulic considerations in site selection are numerous because of the many variations in flow conditions encountered and the many water-related environmental considerations. Section 13.7.4 presents general, good hydraulic practices which should be incorporated at this stage of project development, where practical.

Compliance with the maximum backwater allowance should not be the sole criterion for determining the hydraulic acceptability of a proposed design at a stream crossing. The total performance of a highway-stream crossing system is sensitive to the waterway opening, roadway profile and elevation, pier location and orientation, environment considerations, highway skew and stream morphology. The total system will have significant effects on velocities, flow distribution, stream environment, scour, risks and construction costs. These are discussed in Section 13.7.4.

13.7.4.1 Environmental Considerations

Many aspects of the environmental assessment with respect to the site are also related to the hydraulic design of a stream crossing. These include the effects on the aquatic life in the stream; effects on other developments, such as a domestic or irrigation water supply intake; and the effects on floodplains.

Biological considerations at the site include the effects on habitat and ecosystems in the floodplain and the effects on aquatic ecosystems in the stream and wetlands. Some of the factors to consider include the cost to replace lost marsh or wetland areas; circulation of fresh water in marshes; and the feasibility of providing mitigating measures for the loss of invertebrate populations.

The preservation of wetlands must also be considered in the design of a stream crossing. The importance of wetlands is recognized because of their high productivity of food and fiber; beneficial effects on flooding, pollution and sediment control; and the wildlife habitat they support. Stream crossings must be located and designed so that important wetlands will not be destroyed or their value diminished unnecessarily.

It should also be noted that the evaluation of the typical hydraulic engineering aspects of bridge design are interrelated with environmental impacts. These include the effects of the crossing on velocities, water surface profiles, flow distribution, scour, bank stability, sediment transport, aggradation and degradation of the channel, and the supply of sediment to the stream or water body.

The environmental process for stream crossing projects may also precipitate the need for several State and Federal permits and approvals which are water related. These include:

1. the Stream Preservation Act Permit,
2. Section 402 "Temporary Erosion Control Permit,"
3. U.S. Army Corps of Engineers Section 404 Permit,
4. approvals from U.S. Fish and Wildlife, and

5. approvals related to floodplain encroachments.

The bridge designer should consider the future requirements for these permits and approvals in the preliminary bridge design stage.

13.7.4.2 Stream Types

Section 13.7.2 defines the three basic types of streams — braided, straight and meandering.

Hydraulic analysis of braided streams is extremely difficult because of the inherent instability and unpredictable behavior of such streams. Constricting a braided channel into one channel or placing roadway fill between subchannels may change sediment transport capacity at some locations. Where practical, an alternative crossing site at a reach of stream which is not braided should be selected.

A straight reach of stream channel in an otherwise meandering stream may be viewed as a transient condition. Aerial photographs and topographic maps should be examined for evidence of past locations of the channel and of tendencies for meanders to form in the straight reach.

For meandering streams, the concave bank of a bend (i.e., the bank with the longer radius of curvature) presents the greatest hazard to highway facilities because the stream attacks that bank. The design of crossings at bends is complex because it is difficult to predict flood flow distribution. The stream is usually deeper at that bank, velocities are higher and the water surface is superelevated. The location of a structure in the overbank area may encourage a cutoff and, if the bend system is moving, approach fills and abutments will be subjected to attack as the bend moves downstream.

13.7.4.3 Roadway Alignment

The horizontal and vertical roadway alignment at the bridge will impact the hydraulic performance of the bridge. For example, the accumulation of debris or ice on the upstream side of the structure can increase the effective depth of the superstructure, impose larger hydraulic forces on the bridge superstructure and increase scour depths. Because no relief from these forces is afforded, crossings on zero gradients and in sag vertical curves are more vulnerable than those with profiles which provide an alternative to forcing all water through the bridge waterway.

13.7.4.4 Location of Waterway Openings

The choice of a location for a waterway opening at a stream crossing site with limited floodplain widths is not difficult because it is readily apparent that one opening will suffice. The location of waterway openings is more complex for designs for rare floods and at sites with extensive floodplains. An auxiliary opening(s) may be warranted at these sites.

Several factors influence decisions on the location of the waterway opening to provide for flood passage. These include local drainage, the possibility of causing a cutoff in a meander bend, other transportation facilities in the vicinity, floodplain use, and the horizontal and vertical alignment of the highway.

Basic objectives in choosing the location(s) of auxiliary opening(s) include maintenance of flow distribution and flow directions as practical, provision for relatively large flow concentrations in the floodplains, avoidance of diversion of floodplain flow along roadway embankments for long distances, and considerations of backwater and scour damage to the highway and other property. Site conditions, economics, budgetary constraints, and the horizontal alignment of the highway may limit the extent to which these objectives can be met.

13.7.4.5 Pier Location/Shape

The number of piers in a channel should be limited to a practical minimum, and piers in the channel of small streams should be avoided. Piers properly oriented with the flow do not contribute significantly to bridge backwater, but they do contribute to general scour. In some cases, severe scour may develop immediately downstream of bridges because of eddy currents and because piers occupy a significant area in the channel.

Piers should be aligned with flow direction at flood stage to minimize the opportunity for drift to be trapped, to reduce the contraction effect of piers in the waterway, to minimize ice forces and the possibility of ice dams forming at the bridge, and to minimize backwater and local scour. Pier orientation is difficult where flow direction changes with stage or time. Circular piers, or some variation, are probably the best alternative if orientation at other than flood stage is critical.

Piers located on a bank or in the stream channel near the bank are likely to cause lateral scouring of the bank. Piers located near the stream bank in the floodplain are vulnerable because they can cause bank scour. They are also vulnerable to failure from undermining by meander migration. Piers which must be placed in locations where they will be especially vulnerable to scour damage should be founded at elevations safe from undermining.

Pier shape is also a factor in local scour. A solid pier will not collect as much debris as a pier bent or a multiple-column bent. Rounding or streamlining the leading edges of piers helps to decrease the accumulation of debris and reduces local scour at the pier.

Where ice is a problem, piers are armored and may be battered to facilitate breaking up ice floes which otherwise would crush against the leading edge of the pier.

13.8 ENVIRONMENTAL ISSUES

13.8.1 General

Environmental Services will perform the environmental studies for the proposed bridge project. Section 3.1.2 discusses the necessary coordination between Environmental Services and the Bridge Bureau on the following:

1. NEPA/MEPA requirements;
2. Sections 4(f), 6(f) and 106;
3. mitigation;
4. early coordination;
5. permits/approvals;
6. hazardous waste;
7. erosion control during construction; and
8. wetland mitigation.

Chapter Two documents the project development process for bridge projects when the Bridge Bureau is the lead unit for project development. The project networks illustrate the timing of the interaction between Environmental Services and the Bridge Bureau in evaluating the environmental impacts of the proposed project. As illustrated in Chapter Two, most of the coordination on the above issues occurs after the structure type selection. However, the bridge designer must anticipate and evaluate the likely environmental consequences (both procedural and technical) when selecting the structure type, configuration and size.

13.8.2 Environmental Procedures

The Engineering Bureau within Environmental Services is responsible for ensuring that all MDT projects comply with the National Environmental Policy Act (NEPA) and the Montana Environmental Policy Act (MEPA). The key element in this process is to identify the project's environmental Class of Action based on the expected project environmental impacts. Three classifications exist:

1. Categorical Exclusion (CE). A category of actions which do not individually or cumulatively have a significant effect on the human environment for which, therefore, neither an Environmental Assessment nor an Environmental Impact Statement is required.

2. Environmental Assessment (EA). A document that serves to briefly provide sufficient evidence and analysis for determining whether to prepare an Environmental Impact Statement or a Finding of No Significant Impact (FONSI).

3. Environmental Impact Statement (EIS). A detailed written statement, prepared for major Federal actions significantly affecting the quality of the human environment, which discusses the environmental impact of the proposed action; any adverse environmental effects which cannot be avoided should the proposal be implemented; alternatives to the proposed action; the relationship between local short-term uses of man's environment and the maintenance and enhancement of long-term productivity; and any irreversible and irretrievable commitments of resources which would be involved if the proposed action should be implemented.

In general, the process for an EA is more involved than a CE, and an EIS is more involved than a CE or EA. The environmental process involves many other activities in addition to the Class of Action determination (e.g., early coordination, identifying cooperating agencies, public involvement, selecting and evaluating alternatives). Section 1.3.3 provides a summary of the functions and responsibilities of Environmental Services. The bridge designer should contact Environmental Services for more information on environmental procedures.

13.8.3 Environmental Impacts

In general, any bridge project should, within reason, attempt to minimize the environmental impacts, especially in sensitive areas (e.g., wetlands). The Resources Bureau within Environmental Services is, in general, responsible for identifying all environmental

resources within the proposed project limits and for evaluating the potential project impacts on these resources. The following sections briefly discuss potential environmental impacts precipitated by a bridge project.

13.8.3.1 Water Related

Any bridge over a stream or other water resource has the potential for water-related environmental impacts. These include fish habitat, other aquatic life, water quality, wetlands, floodplains and sediment transport. Section 13.7.4 "Good Hydraulic Practices" provides more discussion on environmental considerations with respect to bridges over water resources.

13.8.3.2 Historic Bridges

Based on Section 106 of the National Historic Preservation Act of 1966 (as amended), MDT must consider the effects of the project on properties included in or eligible for inclusion in the National Register of Historic Places (NRHP). Where such properties will be affected, the Advisory Council on Historic Preservation (ACHP) must be afforded a reasonable opportunity to comment on the undertaking. MDT must implement special efforts to minimize harm to any property on or eligible for the NRHP that may be adversely affected by the proposed project. The mitigation is accomplished through written agreements among MDT, the ACHP and the Montana State Historic Preservation Officer (SHPO).

MDT has identified those historic bridges within the State that are on or eligible for the NRHP. See Figures 13.8A and 13.8B. MDT must comply with the Section 106 procedures for any work on these bridges or bridge work which impacts these bridges.

13.8.3.3 Hazardous Waste

The Hazardous Waste Bureau within Environmental Services is responsible for identifying and evaluating hazardous waste sites and for determining the needed mitigation measures. Three specific types of hazardous waste which may require treatment for a bridge project include:

1. Paint Removal. Removal of lead-based paint from an existing bridge.
2. Timber Removal. Salvaging or disposing of timber from an existing bridge.
3. Plates. Asbestos blast plates on railroad overpasses.

13.8.3.4 Construction

Restrictions on contractor access in environmentally sensitive areas should be established and negotiated early in project development.

13.8.3.5 Local Considerations

The project may be subjected to local environmental considera-tions (e.g., restrictions may be imposed on the areas in which the contractor may work).

13.8.3.6 Other Environmental Impacts

Occasionally, a proposed bridge project may precipitate other environmental impacts. These include Section 4(f), Section 6(f), Section 106 (other than historic bridges), threatened and endangered species and the need for a TERO (Tribal Employment Rights Office) Agreement. Contact Environmental Services for more information.

BRIDGE	MDT ID NUMBER	SMITHSONIAN #	COUNTY	DOC
Browne's Bridge		24BE526	Beaverhead	1916
Beaverhead River (Selway)	L01279000+04001	24BE1738	Beaverhead	1916*
Big Horn River	Abandoned	24BH2464	Bighorn	1911
Milk River (w. of Dodson)	L03068002+05001	24BL1209	Blaine	1916
Milk River (e. of Harlem)	L03068000+05001	24BL1204	Blaine	1914
Clark's Fork at Fromberg	L05307000+07001	24CB1223	Carbon	1914*
Clark's Fork (s. of Belfry)	Abandoned		Carbon	1925
Missouri River (10th Street Bridge)	Non-system	24CA308	Cascade	1920
Milwaukee Rd O'pass/Great Falls	U05217001+05401	24CA331	Cascade	1914
Sand Coulee Cr. (s. of Great Falls)	S00226001+05191	24CA258	Cascade	1937
Heppler Coulee (w. of Simms)	L07561002+00001	24CA256	Cascade	1934
Simms Creek (w. of Simms)	L07561004+04001	24CA257	Cascade	1934
Missouri River at Fort Benton	City-owned	24CH335	Chouteau	1888
Yellowstone River at Fort Keogh	N/A	24CR668	Custer	1902
Tongue River at Miles City	L09054000+01001	24CR679	Custer	1897*
Cottonwood Creek	L09004003+01001	24CR643	Custer	1928
Cottonwood Creek (s. of Joe)	S00242005+05001	24FA2321	Fallon	1934
Big Spring Creek Bridge	Relocated by county	24FR824	Fergus	1908
Flathead River (Old Steel Bridge)	L15091000+05001	24FH463	Flathead	1894*
Flathead River at Columbia Falls	Abandoned	24FH464	Flathead	1911
Gallatin River (Nixon Bridge)	L16216002+02001	24GA393	Gallatin	1891
Gallatin River (Cameron Bridge)	N/A	24GA829	Gallatin	1891
Jefferson River	L16201000+07001	24GA831	Gallatin	1894
St. Mary River at Babb	L18224002+08001	24GL186	Glacier	1915
Musselshell River	L19217000+03001	24GV144	Gold. Valley	1900
Boulder River (Red Bridge)	L22304000+03001	24JF479	Jefferson	1899*
Dearborn River High Bridge	L25300009+00001	24LC130	L&C	1897
Missouri River at Wolf Creek	L25003011+00001	24LC131	L&C	1933
Missouri River at Craig	L25013000+03001	24LC3001	L&C	1903*
10 Mile Cr. Helena (Williams St.)	L25549000+01001	24LC128	L&C	1894
Little Prickly Pear Creek	L25233000+03001	24LC127	L&C	1897
Ten Mile Creek (N. Montana Ave.)	U05809002+09631	24LC1546	L&C	1933*
Flat Creek (se. of Augusta)	P00009023+05901	24LC767	L&C	1931
Marias River (Pugsley Bridge)	L26038005+01001	24LT76	Liberty	1951
Kootenai River at Troy	L27411000+01001	24LN64	Lincoln	1912
Blaine Spring Creek Bridge	S00249007+05001	24MA780	Madison	1892
Madison River (Varney Bridge)	S00249007+08001	24MA779	Madison	1897
Big Hole River Overflow Bridge	L29141015+07001	24MA413	Madison	1907*
Jefferson River at Silver Star	S00422000+01001	24MA412	Madison	1913*
Wisconsin Creek	P00029037+02091	24MA714	Madison	1938
Cherry Creek	P00029053+06001	24MA795	Madison	1935
Clark Fork River (Van Buren Street)	N/A	24MO248	Missoula	1895
Orange Street Bridge	U08107001+05401	24MO553	Missoula	1936*
Milwaukee Road Overpass	U08107001+04101	24MO554	Missoula	1935*

MONTANA HIGHWAY BRIDGES ON OR ELIGIBLE FOR THE NATIONAL REGISTER OF HISTORIC PLACES (June 2000)

Figure 13.8A

BRIDGE	MDT ID NUMBER	SMITHSONIAN #	COUNTY	DOC
Stockpass (w. of Musselshell)	P00014182+04001	24ML245	Musselshell	1937*
Carter Bridge	S00540031+06621	24PA777	Park	1921
Milk River at Wagner	S00363001+03001	24PH3216	Phillips	1910*
Assiniboine Creek	S00242006+00001	24PH2669	Phillips	1936
Como Bridge	L41600000+01001	24RA523	Ravalli	1917
Wolf Point Bridge	Non-system	24RV438	Roosevelt	1930
Yellowstone River at Reed Point	L48083000+07001	24ST216	Stillwater	1911*
Stillwater River (Kern's Crossing)	L48128000+04001	24ST215	Stillwater	1902
Dry Wash (n. of Augusta)	P00057217+07911	24TT120	Teton	1936
Drainage	P00009059+01911	24TT121	Teton	1936
Drainage	P00003057+07501	24TT122	Teton	1929
Drainage	P00003059+01911	24TT123	Teton	1929
Jones Coulee (s. of Pendroy)	P00003059+07801	24TT125	Teton	1929
Buckingham Coulee	S00311019+00001	24TE44	Treasure	1936*
Milk River at Tampico	L53507000+03001	24VL722	Valley	1911*
Yellowstone R. at Pompeys Pillar	S00568000+09601	24YL784	Yellowstone	1915

***Bridge has been programmed for replacement.**

MONTANA HIGHWAY BRIDGES ON OR ELIGIBLE FOR THE NATIONAL REGISTER OF HISTORIC PLACES (June 2000)

Figure 13.8A
(Continued)

BRIDGE	MDT ID NUMBER	SMITH. #	COUNTY	DOC	ON
Frying Pan Gulch	L01311005+01001	24BE548	Beaverhead	1929	X
Poindexter Slough 2	FWP	24BE1401	Beaverhead	1929	
Limekiln Canyon	L01309014+09001	24BE537	Beaverhead	1931	X
Poindexter Slough	S00222002+07511	24BE538	Beaverhead	1936	X
UPRR Overpass	S00222002+08611	24BE539	Beaverhead	1936	X
Two Leggin Canal	L02313021+06001	24BH1981	Bighorn	1931	X
Big Horn R/Hardin	S00348000+05101	24BH376	Bighorn	1942	X
Little Big Horn R. 3	S00451008+06371	24BH2872	Bighorn	1955	X
Lodge Grass Creek	S00451015+07741	24BH2873	Bighorn	1955	X
Little Big Horn R. 1	I00090511+06051	24BH1642	Bighorn	1956	X
Little Big Horn R. 2	I00090517+05021	24BH1643	Bighorn	1956	X
Lodge Creek Bridge	P00001404+05791	24BL1050	Blaine	1942	X
Allen/Wills Ditch	County	24CB1508	Carbon	1911	
Bluewater Creek	L05302008+06001	24CB1309	Carbon	1913	
East Bench Canal	Private	24CB1356	Carbon	1915	
West Canal	Private	24CB713	Carbon	1915	
E. Rosebud Cr/Roscoe	L05503000+01001	24CB1310	Carbon	1915	
Willow Creek Bridge	P00023126+04531	24CT593	Carter	1955	X
Belt Creek	P00060040+03761	24CA641	Cascade	1921	X
Novak Creek Bridge	L07604005+00001	24CA394	Cascade	1931	X
Fort Shaw Canal	L07561003+04001	24CA395	Cascade	1934	X
GNRR Overpass	P00080000+02091	24CH983	Chouteau	1953	X
Poplar River Bridge	S00248043+03531	24DN7	Daniels	1940	X
Bad Route Creek	L11109020+03001	24DW423	Dawson	1921	
Warm Springs Creek	L12163000+01001	24DL485	Deer Lodge	1912	
Silver Bow Creek	L12079000+02001	24DL707	Deer Lodge	1928	X
Opportunity Drainage	L12146002+06001	24DL705	Deer Lodge	1931	X
Clark Fork River	L12146001+01001	24DL706	Deer Lodge	1931	X
Willow Creek Bridge	L12146001+02001	24DL680	Deer Lodge	1931	X
Big Spring Creek	U07104001+00801	24FR801	Fergus	1920	
Milwaukee Road Overpass	P00057079+06521	24FR803	Fergus	1936	X
Abbot Creek	L15689000+04001	24FH498	Flathead	1931	X
GNRR Overpass	P00001167+05251	24FH441	Flathead	1956	X
Baker Creek	S00205013+00791	24GA800	Gallatin	1921	
Drainage	S00205008+03441	24GA1069	Gallatin	1931	X
NPRR Overpass/Logan	S00205005+00671	24GA1070	Gallatin	1934	X
Gallatin River	P00050049+08111	24GA1511	Gallatin	1952	X
S. Fk. Cut Bank Creek	P00058012+09171	24GL236	Glacier	1927	X
S. Fk. Milk River	P00058021+00281	24GL237	Glacier	1928	X
Two Medicine River	P00001210+01961	24GL935	Glacier	1941	X
Fred Burr Creek Bridge	P00019035+03141	24GN844	Granite	1931	X
Big Sandy Creek	P00001376+02951	24HL1120	Hill	1954	X
Cataract Creek Bridge	L22099008+00001	24JF547	Jefferson	1933	X
Boulder River Bridge	L22099004+03001	24JF1599	Jefferson	1933	X

REINFORCED CONCRETE BRIDGES ELIGIBLE FOR THE NATIONAL REGISTER OF HISTORIC PLACES
(June 2000)
Figure 13.8B

BRIDGE	MDT ID NUMBER	SMITH. #	COUNTY	DOC	ON
Whitetail Creek Bridge	P00069000+07621	24JF767	Jefferson	1955	X
Coyote Creek	L23204002+01001	24JT299	Judith Basin	1934	
Mud Creek	L24009003+08001	24LA207	Lake	1920	
Mission Creek	M24100000+00101	24LA226	Lake	1929	
Elk Creek Overflow	L25309000+01001	24LC1599	L & C	1920	
Prickly Pear Creek	M25037000+00301	24LC504	L & C	1920	X
Wegner Creek	L25003016+05001	24LC133	L & C	1930	X
Sheep Creek	L25003005+02001	24LC1157	L & C	1933	X
McLeod Creek Bridge	Abandoned	24LC1226	L & C	1933	
Elk Creek Bridge	L25400000+02001	24LC550	L & C	1935	X
Curley Creek Bridge	L27043000+03001	24LN1599	Lincoln	1930	
Rock Creek Bridge	L31179000+05001	24MN302	Mineral	1931	
12 Mile Creek Bridge	L31179001+09001	24MN244	Mineral	1931	
Rattlesnake Creek	U08110000+05751	24MO706	Missoula	1932	X
Vine Street Bridge	M32081000+01071	24MO522	Missoula	1945	
Ferry Creek	L34003001+07001	24PA1077	Park	1921	
NPRR Overpass	P00059000+00611	24PA1137	Park	1954	X
GNRR Overpass	P00001459+03301	24PH3177	Phillips	1936	X
Little Powder River	L38665004+05001	24PR1837	Powder River	1947	
Conley St. /Deer Lodge	M39031000+00401	24PW608	Powell	1914	
Racetrack Creek Bridge	L39242006+06001	24PW697	Powell	1930	X
Cottonwood Creek	S00275001+00951	24PW607	Powell	1933	X
Big Blackfoot River	L39001011+04001	24PW700	Powell	1954	
Yellowstone River	L40114001+05001	24PE618	Prairie	1945	X
Powder River	L40004006+02001	24PE1810	Prairie	1945	X
Tin Cup Creek	County	24RA521	Ravalli	1915	
Wolf Creek	P00001588+08561	24RV655	Roosevelt	1956	X
Smoke Creek Bridge	L43205005+09001	24RV295	Roosevelt	1950	
Armells Creek	L44311005+07001	24RB500	Rosebud	1932	X
Cabinet Gorge/Heron	L45025001+00001	24SA471	Sanders	1952	
Silver Bow Cr/Nissler	Abandoned	24SB581	Silver Bow	1915	
BPR Overpasses	I00015124+00371	24SB616	Silver Bow	1955	X
Dale Creek	S00419018+02001	24ST286	Stillwater	1945	X
Stillwater River/Nye	S00419019+09001	24ST289	Stillwater	1945	X
Big Ditch Bridge	L48245011+05001	24ST275	Stillwater	1955	X
American Fork Bridge	P00045033+00271	24WL155	Wheatland	1942	X

REINFORCE CONCRETE BRIDGES ELIGIBLE FOR THE NATIONAL REGISTER OF HISTORIC PLACES
(June 2000)
Figure 13.8B

(Continued)

13.8.4 Permits/Approvals

A proposed bridge project may precipitate the need for one or more environmental permits or approvals. Except for floodplains (which are the responsibility of the Hydraulics Section), Environmental Services is responsible for coordinating with the applicable Federal or State agency and acquiring the permit or approval. This will require considerable coordination with the Bridge Bureau. The following sections briefly discuss these permits/approvals.

13.8.4.1 Montana SPA Permit

The Montana Stream Preservation Act (SPA) requires that MDT secure a 124 SPA Permit for work in *any* named stream/waterbody or tributary to a stream, lake, pond or other waterbody. The requirement for this Permit extends to ephemeral streams if the work proposed is within 30 m of a named or unnamed perennial or intermittent tributary to a stream, lake, pond or other waterbody. The Permit requirement, however, does not extend to wetlands.

Because of the environmental sensitivities associated with all stream crossings, early coordination with the regulatory agencies is essential on bridge projects. Do not expend man-hours designing a bridge that does not have the necessary approvals from the regulatory agencies. The project development networks in Chapter Two illustrate how the SPA Permit interacts with the bridge process.

13.8.4.2 U.S. Army Corps of Engineers Section 404 Permit

The Section 404 Permit is required for the discharge of dredge or fill material into any waters of the United States, including wetlands. The purpose of Section 404 is to restore and maintain the chemical, physical and biological integrity of the Nation's waters through the prevention, reduction and elimination of pollution.

For identification, the "waters of the United States" includes all wetlands and areas within a blue solid line or a blue dashed line on the USGS quadrangle maps. Each river, stream, creek, intermittent tributary, pond, impoundment, lake or wetlands is considered part of the waters of the United States. More commonly, the waters of the United States are usually interpreted as any named stream or waterbody and unnamed intermittent tributaries that have "defined gravel bottoms."

Wetlands are also described as bogs, marshes, sloughs and swamps. Floodplains, or areas where water stands on, at or near the groundline, may be considered suspected wetlands. Guidelines as established by the U.S. Army Corps of Engineers indicate that a wetland must have all of the following characteristics:

1. a preponderance of water-tolerant plants;
2. hydric soils; and
3. water on, at or near the surface of the ground during a specified portion of the growing season.

Where Section 404 applies, the project either may be covered by one of the nationwide permits or may require an individual Section 404 Permit. In general, if a project will not involve more than minor water quality impacts, it may be eligible for a nationwide Section 404 Permit. Contact Environmental Services, which has guidelines on nationwide versus individual Permits, for a determination on this issue.

13.8.4.3 Section 401 Water Quality Certification

Pursuant to Section 401 of the Federal Water Pollution Control Act of 1972 (as amended), the Section 401 Water Quality Certification is issued by the Montana Department of Environmental Quality based on regulations issued by the U.S. Environmental Protection Agency. The purpose of the Section 401 Permit is to restore and maintain the chemical, physical

and biological integrity of the Nation's waters through prevention, reduction and elimination of pollution. A Section 401 Certification (or waiver of Certification) is required in conjunction with any Federal permit (e.g., a Section 404 Permit) to conduct any activity which may result in any discharge into waters of the United States.

13.8.4.4 Section 402 NPDES Permit

Pursuant to Section 402 of the Federal Water Pollution Control Act of 1972 (as amended), the Section 402 National Pollutant Discharge Elimination System (NPDES) Permit is issued by the Montana Department of Environmental Quality based on regulations issued by the U.S. Environmental Protection Agency. The purpose of the Section 402 Permit is to restore and maintain the chemical, physical and biological integrity of the Nation's waters through prevention, reduction and elimination of pollution.

13.8.4.5 U.S. Coast Guard Section 10 Permit

Pursuant to Section 10 of the Rivers and Harbors Act of 1899, the Section 10 Permit is issued by the U.S. Coast Guard. The purpose of the Section 10 Permit is to protect and preserve the navigable waterways of the United States against any degradation in water quality. The Permit is required for structures or work (other than bridges or causeways) affecting a navigable waterway (tidal or non-tidal). Examples of work include dredging, channelization and filling. In Montana, the following are navigable waterways:

1. the Missouri River,
2. the Yellowstone River below Emigrant, and
3. the Kootenai River above Jennings.

13.8.4.6 Floodplains Encroachment

Pursuant to Executive Order 11988 "Floodplain Management," MDT must seek approval from the Federal Emergency Management Agency (FEMA) for any Federally funded/regulated project which produces a significant floodplain encroachment. If a project will have a significant floodplain encroachment, the project will require either an Environmental Assessment (EA) or Environmental Impact Statement (EIS). A proposed action which includes a significant floodplain encroachment will not be approved unless FHWA finds (pursuant to 23 CFR650A) that the proposed action is the only practical alternative.

The intent of Executive Order 11988 is to:

1. encourage a broad unified effort to prevent uneconomic, hazardous or incompatible use and development of floodplains;

2. avoid significant encroachments, where practical;

3. minimize impacts of actions which adversely affect base floodplains;

4. restore and preserve the natural and beneficial floodplain values that are adversely impacted by proposed actions;

5. avoid support of incompatible floodplain development; and

6. be consistent with the intent of the Standards and Criteria of the National Flood Insurance Program (NFIP), where appropriate.

The Hydraulics Section is responsible for determining that the bridge design is consistent with the regulations promulgated by FEMA and by FHWA (23 CFR 650A) and, where necessary, the Section prepares a Floodplain Study and/or a Floodplain Finding.

13.8.4.7 Other Montana State Permits

In addition to the SPA Permit, the following State permits may also be required on bridge projects:

1. Section 402 Montana Pollutant Discharge Elimination System (MPDES) Authorization. This is also commonly called the stormwater discharge permit. This authorization is required for projects that have over 0.4 ha of "disturbance" within 30 m of "State waters," or over 2 ha of "disturbance" anywhere in Montana. "State waters" are considered to be *any* named stream or waterbody, including irrigation systems and unnamed perennial or intermittent tributaries to a stream lake, pond or other waterbody, including wetlands. Ephemeral streams qualify if the proposed work is within 30 m of a named or unnamed perennial or intermittent stream. Note that "State waters" do not include streams on Indian Reservations. National Pollutant Discharge Elimination System (NPDES) and Stormwater Pollution Prevention Plan (SWPP) authorization must be secured for work proposed on an Indian Reservation.

2. Aquatic Lands Permits (ALPO). The ALPO Permit is needed for work in *any* named stream or waterbody or any unnamed perennial or intermittent tributary to a stream, lake, pond or other waterbody on certain Indian Reservations. Ephemeral streams also need this Permit if work occurs within 30 m of a named or unnamed perennial or intermittent stream or tributary. The enabling ordinance for this Permit is the Aquatic Lands Conservation Ordinance (ALCO). This is also called an ALCO Permit. As of this date, the only tribal governments to enact this ordinance have been the Blackfeet and the Flathead.

3. Montana Asbestos Abatement Project Permit and National Emission Standards for Hazardous Air Pollutant Demolition/ Renovation Notification. These requirements apply to structures with known asbestos treatments.

13.9 STRUCTURE TYPE, SIZE AND LOCATION

The intent of the preliminary layout design is to select a structure type, size and location that addresses all of the aspects discussed in Chapter Thirteen of the **Montana Structures Manual**. The preliminary layout design is reviewed by other design units (Road Design, Hydraulics, Geotechnical, etc.), Environmental Specialists, the District and Resource Agencies (Fish, Wildlife and Parks, Department of Environmental Quality, Corps of Engineers). Once concurrence has been received from all areas, it is risky to change the layout. Changes can result in higher design cost and time delays for the project.

The preliminary layout design should be summarized in a Structure Type, Size and Location Report. The Report should document all issues considered in the design of the layout and include structural design parameters.

Items to be included in the Structure Type, Size and Location Report include:

1. hydraulics,
2. environmental,
3. roadway alignment,
4. bridge length and width,
5. bridge beam type,
6. geometrics,
7. riprap,
8. fit of bridge to site,
9. proposed substructure,
10. geotechnical/core logs,
11. proposed foundation site,
12. seismic, and
13. alternative structure types.

For guidance on the content and format of the Structure Type, Size and Location Report, use the information in Section 4.1.3 on the Design Parameters Report.

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Chapter Fourteen
LOADS AND ANALYSIS

14.1 GENERAL

14.1.1 Introduction

Articles 1, 3 and 4 of the LRFD Bridge Design Specifications discuss various aspects of loads and analysis. Unless noted otherwise in Chapter Fourteen of the **Montana Structures Manual**, the LRFD Specifications apply to loads and analysis in Montana. Chapter Fourteen also presents additional information on MDT practices.

14.1.2 Types of Loads (Definitions)

1. Permanent loads. Loads which are always present in or on the bridge and do not change in magnitude during the life of the bridge. Specific permanent loads include:

   a. Gravitational Dead Loads:

      DC – dead load of all of the components of the superstructure and substructure, both structural and non-structural.

      DW – dead load of additional wearing surfaces and any utilities crossing the bridge.

      EL – accumulated lock-in, or residual, force effects resulting from the construction process, including the secondary forces from post-tensioning.

   b. Earth Pressures:

      EH – horizontal earth pressure.

      EV – vertical earth pressure from dead load of earth fill.

      ES – earth pressure from dead load of an earth surcharge.

      DD – downward skin friction on the sides of piles from settlement.

2. Transient loads. Loads which are not always present in or on the bridge or change in magnitude during the life of the bridge. Specific transient loads include:

   a. Live Loads:

      LL – vertical gravity loads due to vehicular traffic on the roadway.

      IM – dynamic load allowance due to moving vehicles, traditionally called impact.

      LS – earth pressure from vehicular traffic on the ground surface.

      BR – horizontal vehicular braking force.

      CE – horizontal centrifugal force from vehicles on a curved roadway.

   b. Water Loads:

      WA – pressure due to differential water levels, stream flow or buoyancy.

   c. Wind Loads:

      WS – horizontal and vertical pressure on superstructure or substructure due to wind.

      WL – horizontal pressure on vehicles due to wind.

d. Extreme Events:

EQ – horizontal loads due to earthquake ground motions.

CT – horizontal impact loads on piers or abutments due to vehicles or trains.

CV – horizontal impact loads due to aberrant ships or barges.

IC – horizontal static and dynamic forces due to ice action.

e. Superimposed Deformations:

TU – uniform temperature change due to seasonal variation.

TG – temperature gradient due to exposure of the bridge deck to solar radiation while the superstructure under the deck is shaded from the sun.

SH – differential shrinkage between different concretes or concrete and non-shrinking materials, such as metals and wood.

CR – creep of concrete or wood.

SE – the effects of settlement of substructure units on the superstructure.

FR – frictional forces on sliding surfaces from structure movements.

14.1.3 Limit States

Reference: LRFD Article 1.3.2.1

In the LRFD Specifications, the traditional design criteria have been grouped together with the groups termed limit states. The various limit states then have load combinations assigned to them. Components and connections of a bridge are designed to satisfy the basic LRFD equation for all limit states:

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n \qquad \text{(Equation 14.1.1)}$$

Where:

γ_i = load factor
Q_i = load or force effect
ϕ = resistance factor
R_n = nominal resistance
η_i = load modifier as defined in LRFD Equations 1.3.2.1-2 and 1.3.2.1-3

The left-hand side of Equation 14.1.1 is the sum of the factored load (force) effects acting on a component; the right-hand side is the factored nominal resistance of the component for the effects. The Equation must be satisfied for all the applicable limit state load combinations.

The load modifier η_i is either the product of, or the reciprocal of, the product of the factors η_D, η_R and η_i relate to ductility, redundancy and operational importance. Its location on the load side of the Equation may seem counter-intuitive because it seems more related to resistance than to load. It is on the load side for a logistical reason. When it modifies a maximum load factor, η_i is the product of the factors; when it modifies a minimum load factor, it is the reciprocal of the product. These factors are based on a 5% stepwise positive or negative adjustment, reflecting unfavorable or favorable conditions. They are somewhat arbitrary; their significance is in their presence in the LRFD Specifications and not necessarily in the accuracy of their magnitude.

MDT uses η_i values of 1.00 for all limit states.

14.1.4 Load Factors and Combinations

Reference: LRFD Article 3.4.1

LRFD Table 3.4.1-1 provides the load factors for all of the limit state load combinations of the Specifications. The significance of the strength limit state load combinations can be simplified as follows:

1. Strength I Load Combination. This load combination represents random traffic and the heaviest truck to cross the bridge in its 75-year design life. During this live-load

event, a significant wind is not considered probable.

2. Strength II Load Combination. This load combination represents an owner-specified permit load model. This live-load event will have less uncertainty than random traffic and thus a lower live-load load factor.

3. Strength III Load Combination. This load combination represents the most severe wind during the bridge's 75-year design life. During this extreme wind event, no significant live load would cross the bridge.

4. Strength IV Load Combination. This load combination represents an extra safeguard for long-span bridge superstructures (i.e., where the unfactored dead load exceeds seven times the unfactored live load). With long-span bridges, the live load becomes less significant. Thus, the only significant load factor would be the 1.25 dead-load maximum load factor. For additional safety, and based solely on engineering judgment, the load factor for DC has been arbitrarily increased to 1.5.

5. Strength V Load Combination. This load combination represents the simultaneous occurrence of an "average" live-load event and an "average" wind event with "average" load factors of 1.35 and 0.4, respectively.

Unlike the strength limit state load combinations, the service limit state load combinations are, for the most part, material-dependent. The Service I load combination is applied for the checking of cracking of reinforced concrete components and compressive stresses in prestressed concrete components. The Service II load combination is applied for checking permanent deformations of compact steel sections and slip of slip-critical (i.e., friction-type) bolted steel connections. The Service III load combination is applied for checking tensile stresses in prestressed concrete components. Finally, the Service I load combination is also used to calculate deformations and settlements of superstructure and substructure components.

The extreme-event limit state load combinations are applied for earthquakes (Extreme Event I), and various types of collisions (vessel, vehicular or ice) one at a time (Extreme Event II). The extreme-event limit states are different from the strength limit states because the event for which the bridge and its components are designed has a greater return period than the 75-year design life of the bridge.

The fatigue-and-fracture limit state load combination, although strictly applicable to all types of superstructures, only affects the proportions of a limited number of steel superstructure components.

In LRFD Table 3.4.1-1, the load factors for all of the permanent loads, shown in the first column of load factors, are represented by the variable γ_P. This reflects the fact that the strength and extreme-event limit states load factors for the various permanent loads are not constants, but they can have two extreme values. These two extreme values of the various permanent load factors, maximum and minimum load factors, are given in LRFD Table 3.4.1-2. Permanent loads are always present on the bridge, but the nature of variability is that the actual loads may be more or less than the nominal specified design values. Therefore, maximum and minimum load factors reflect this variability. The application of these permanent load factors is discussed in Section 14.2.

The load factors for the superimposed deformations for the strength limit states also have two specified values: a load factor of 0.5 for the calculation of stress and a load factor of 1.2 for the calculation of deformation. The greater value of 1.2 is used to calculate unrestrained deformations, such as a simple span expanding freely with rising temperature. The lower value of 0.5 for the elastic calculation of stress reflects the inelastic response of the structure due to restrained deformations. For example, one-half of the temperature rise would be used to elastically calculate the stresses in a constrained structure. Using 1.2 times the temperature rise in an elastic calculation would overestimate the stresses in the structure which resists the temperature inelastically through

redistribution of the elastic stresses. The application of these load factors for the superimposed deformation is discussed in Section 14.4.5.

14.2 PERMANENT LOADS

14.2.1 General

Reference: LRFD Article 3.5

The LRFD Specifications specify seven components of permanent loads, which are either direct gravity loads or caused by gravity loads. New in this group is downdrag, "DD," which is a negative load in driven piles or drilled shafts as a result of consolidation of soil through which they are driven or drilled. Prestressing is considered, in general, to be part of resistance of a component and has been omitted from the list of permanent loads in Section 3 of the Specifications. However, when designing anchorages for prestressing tendons, the prestressing force is the only load effect, and it should appear on the load side of the LRFD Equation.

As discussed previously in Section 14.1.4 and shown in Table 3.4.1-2 of the LRFD Specifications, there are maximum and minimum load factors for the permanent loads. The maximum or minimum permanent-load load factors should be selected to produce the more critical load effect. For example, in continuous superstructures with relatively short-end spans, transient live load in the end span causes the bearing to be more compressed while transient live load in the second span causes the bearing to be less compressed and perhaps lift up. To check the maximum compression force in the bearing, place the live load in the end span and use the maximum DC load factor of 1.25. To check possible uplift of the bearing, place the live load in the second span and use the minimum DC load factor of 0.90.

In superstructure design, maximum permanent-load load factors are used almost exclusively, with the most common exception being uplift of a bearing. Maximum and minimum permanent-load load factors are used routinely for substructure design. The application of these load factors for substructure design is discussed more completely in Section 14.3.

14.2.2 Uplift

Reference: LRFD Article 3.4.1

In the former AASHTO Standard Specifications, uplift was treated as a separate load combination. With the introduction of maximum and minimum load factors in the LRFD Specifications, load situations such as uplift where a permanent load (in this case a dead load) reduces the overall force effect (in this case a reaction) have been generalized. Permanent load factors, either maximum or minimum, must be chosen for each load combination to produce extreme force effects.

Secondary forces from pre- or post-tensioning are included in the permanent load, EL. As specified in LRFD Table 3.4.1-2, a constant load factor of 1.0 should be used for both maximum and minimum load factors.

14.2.3 Deck Slab

Reference: LRFD Article 9.7.3

MDT uses the Traditional Design methodology outlined in Article 9.7.3 of the LRFD Specifications, unless otherwise approved by the Bridge Design Engineer. For bridge deck and slab design requirements, see Chapters 15 and 16 of this **Manual**.

Bridge deck dead load (DL) for design consists of composite and non-composite components.

Non-composite loads include the weight of the plastic concrete, forms and other construction loads typically required to place the deck. Calculate the non-composite DL using the full slab volume including haunch volumes times the unit weight of concrete. Use 24 kN/m^3 to account for concrete weight and construction loads.

Composite loads are applied to combined beam and slab section properties and include the weight of any curb, rail or barrier placed after the deck concrete has hardened. In addition,

include an allowance for a future overlay of 5×10^{-4} MPa over the entire deck area between the rail or curb faces. Future wear should not be included for slabs of buried structures.

14.2.4 Dead Load Distribution

Reference: LRFD Article 4.6.2.2.1

For distribution of the weight of plastic concrete including that of the integrated sacrificial wearing surface, the formwork should be assumed to be simply supported between interior beams and cantilevered over the exterior beams.

Superimposed dead loads (e.g., curbs, barriers, sidewalks, parapets, railings, future wearing surfaces), if placed after the deck slab has cured, may be distributed equally to all girders. In some cases, such as staged construction and heavier utilities, a more accurate distribution of superimposed dead loads is warranted.

14.2.5 Downdrag on Deep Foundations

Deep foundations (i.e., driven piles and drilled shafts) through unconsolidated soil layers may be subject to downdrag, DD. Downdrag is a negative load on the deep foundation as the soil surrounding it consolidates and settles. This additional load is calculated as a skin-friction effect. If a bridge is at a site where downdrag is anticipated, MDT practice is to mitigate instead of quantify. Section 20.3.4 discusses two potential downdrag mitigation methods.

14.3 TRANSIENT LOADS

14.3.1 General

The LRFD Specifications recognize 19 transient loads. Static water pressure, stream pressure, buoyancy and wave action have been integrated as water load, WA. Creep, settlement, shrinkage and temperature (CR, SE, SH, TU and TG) have been elevated in importance to "loads," being superimposed deformations causing force effects. Vehicular braking force, BR, has been increased considerably to reflect the improvements in the mechanical capability of modern trucks.

14.3.2 Vehicular Live Load (LL)

14.3.2.1 General

Reference: LRFD Articles 3.6.1.1, 3.6.1.2 and 3.6.1.3

For short and medium span bridges, which predominate in Montana, vehicular live load is the most important load. The HL93 live-load model is a notional load in that it no longer qualifies as a true representation of actual truck weights. Instead, the force effects (i.e., the moments and shears) due to the superposition of vehicular and lane load are a true representation of the force effects due to actual trucks.

The components for each design lane are:

1. either the familiar MS18 truck, now called the design truck, or a 220-kN tandem, similar to the Alternate Loading, both of the former Standard Specifications; and

2. a 9.5-kN/m uniformly distributed lane load, similar to the lane load of the former Standard Specifications but without any of the associated concentrated loads.

Note that the dynamic load allowance, IM, of 0.33 is applicable only to the constituent axle and wheel loads of the design truck and the design tandems, but not to the uniformly distributed lane load.

The multiple presence factor of 1.0 for two loaded lanes, as given in LRFD Table 3.6.1.1.2-1, is the result of the Specifications' calibration process, which has been normalized relative to the occurrence of two side-by-side, fully correlated, or identical, vehicles. The multiple presence factor of 1.2 for one loaded lane should be used wherever a single design tandem or single design truck or their constituent axle or wheel loads govern, such as in overhangs, decks, etc. The factor for one loaded lane should never be used for fatigue loads.

The LRFD Specifications retain the traditional design lane width of 3.6 m and the traditional spacing of the axles and wheels of the MS18 truck. Both vehicles (the design truck and design tandem) and the lane load occupy a 3.0-m width placed transversely within the design lane for maximum effect. The lane load is no longer an alternative to the truck, but one applied simultaneously to the truck.

The Specifications require that two closely spaced design trucks superimposed on the lane load be applied on adjacent spans of continuous structures for negative moments and reactions. The reduced probability of such an occurrence of fully correlated, or identical, vehicles is accommodated by multiplying the resulting force effects by 0.9. This sequence of highway loading is specified for negative moment and reaction due to the shape of the influence lines for such force effects. It is not extended to other structures or portions of structures because it is not expected to govern for other influence-line shapes.

14.3.2.2 Load Applications

14.3.2.2.1 Use of Two Design Trucks

Reference: LRFD Article 3.6.1.3.1

The combination of the lane load and a single vehicle (either a design truck or a design

tandem) does not always adequately represent the real-life loading by two closely spaced heavy vehicles, interspersed with other lighter traffic. Two design trucks, with a clear distance not less than 15 000 mm between them and with an adjustment factor of 0.90 will approximate a statistically valid representation.

In positioning the two trucks to calculate the negative moment over an internal support of a continuous girder, spans should be at least approximately 2810 mm in length to position a truck in each span's governing position. If the spans are larger than 2810 mm in length, the trucks remain in governing positions but, if they are smaller than 2810 mm, the maximum force effect can only be attained by trial-and-error with either one or both trucks in off-positions (i.e., non-governing positions for each individual span).

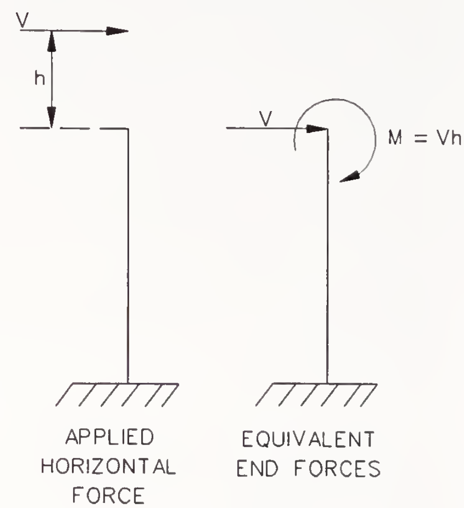
In any case, the moment can be calculated using the influence ordinates directly under each truck's axles.

14.3.2.2.2 Application of Horizontal Superstructure Forces to the Substructure

The transfer of horizontal superstructure forces to the substructure is dependent on the type of superstructure-to-substructure connection. Connections can be fixed, pinned or free for both moment and shear. Although expansion shoes are assumed to be free connections, a horizontal force transfer due to friction should be considered in a conservative nature. Include friction forces where design loads would increase, but neglect friction forces where design loads would decrease.

If the horizontal superstructure force is being applied to the substructure through a pinned connection, there is no moment transfer. Apply the superstructure force to the substructure at the connection.

For a fixed or moment connection, apply the superstructure horizontal force with an additional moment to the substructure as shown in Figure 14.3A. The additional moment is equal to the horizontal force times the distance between the force's line of action and the point of application.



TRANSFER OF HORIZONTAL SUPERSTRUCTURE FORCE TO SUBSTRUCTURE THROUGH MOMENT CONNECTION

Figure 14.3A

14.3.2.3 Fatigue Load

Reference: LRFD Article 3.6.1.4.1

The LRFD Specifications define the fatigue load for a particular bridge component by both magnitude and frequency. The magnitude of the fatigue load consists of a single design truck per bridge with a load factor of 0.75; i.e., an MS13.5 truck. This produces a considerable reduction on the stress range in comparison with the stress ranges of the former Standard Specifications. However, fatigue designs using the LRFD Specifications are virtually identical to those of the Standard Specifications. This is accomplished through an increase in the frequency from values on the order of two million cycles in the Standard Specifications to frequencies on the order of tens and hundreds of millions of cycles. This change to more realistic stress ranges and cycles was made to increase

the designer's understanding of the extreme fatigue lives of steel bridges.

14.3.2.4 Vehicular Centrifugal Force (CE), Vehicular Braking Force (BR) and Wind on Live Load (WL)

Reference: LRFD Articles 3.6.3, 3.6.4 and 3.8.1.3

Vehicular centrifugal forces, vehicular braking forces and wind on live load shall be applied longitudinally or transversely, as appropriate, at a distance of 1800 mm above the roadway's profile grade.

14.3.2.5 Application of Live Load to Piers

Reference: LRFD Article 3.6.1.3.1

To promote uniformity in application of live load to pier bents, hammerhead piers and similar substructures, the following procedure is suggested unless a more exact distribution of loads is used:

1. The live load distribution factor for each girder shall be determined assuming the deck is acting as a simple beam between interior girders and as a cantilever spanning from the first interior girder over the exterior girder.
2. Design lanes shall be placed on the bridge to produce maximum force effect for the component under investigation. The HL93 live load shall be placed within its individual design lane to likewise produce the maximum effect. One, two or more design lanes shall be considered in conjunction with the multiple presence factors of LRFD Table 3.6.1.1.2.-1, as can be accommodated on the roadway width.
3. Two closely spaced design trucks superimposed over the lane load as specified in LRFD Article 3.6.1.3 for negative moment in continuous girders and interior reactions shall be used with a distribution factor derived as discussed above in a line-girder analysis to determine the reaction on interior piers.

14.3.3 Friction Forces (FR)

Reference: LRFD Article 3.13

Section 19.3 discusses friction forces within the context of bearings.

14.3.4 Earthquake Effects

Reference: LRFD Article 3.10

Montana is one of several states in which a high seismic risk exists. The risk is not uniform across the state, with much of the eastern part of the state relatively non-seismic. The probability of high seismic acceleration is predicted only for the Missoula and Butte Districts.

Article 3.10.3 of the LRFD Specifications requires that each bridge be classified according to its Importance Category. In Montana, all bridges are designed as "Other Bridges."

MDT has developed a program to evaluate proposed new bridges in the State highway system to ensure that they meet the AASHTO criteria for seismic design. The Seismic Unit supports the Bridge Design Section in the seismic design and analysis of new bridges. In this capacity, the Unit performs a significant amount of preliminary design work for bridges within the context of addressing seismic vulnerability.

14.3.5 Ice Forces on Piers

Reference: LRFD Article 3.9

During the Field Review or Survey Phase of the project, the Bridge Area Engineer will contact the local landowners, the county and/or local maintenance personnel to determine if ice

problems exist at the bridge site. Discussions should include whether or not ice jams are common, if localized flooding or roadway overtopping occurs, and whether or not the ice at the bridge site is in large floes or broken chunks. If there is an existing bridge, it should be inspected for ice damage. Tree bark may also show scarring from ice damage. This information should help in determining an appropriate ice elevation. The ice force is usually applied at the design high water, unless these investigations or specific site circumstances indicate otherwise. Do not use the reduction in ice forces as suggested by LRFD C3.9.2.3.

For uniformity of State practice, use the following values for thickness of ice "t":

1. "t" = 460 mm for the Missouri River below Loma, the Yellowstone River below Laurel and the Milk River below Malta.

2. "t" = 300 mm for anything else.

Use the following values for pressure:

1. 1.15 MPa for the Missouri River below Loma and the Yellowstone River below Laurel.

2. 0.77 MPa generally statewide, except 0.38 MPa for small streams or rivers that usually do not freeze over in winter or where ice movement would consist mostly of broken fragments and disintegrated ice. This includes larger rivers in the western part of the state such as the Flathead, Kootenai and Clark Fork below Missoula.

These values represent engineering data for design. For the information to be presented in the construction plans, refer to Figure 14.3B. Use the "small stream" reduction factor of 0.5 as allowed by Article 3.9.2.3 for all streams with a width less than 90 m at the mean water level. In the absence of better information, using the width of the channel at Q_2 flows instead of the mean water elevation will result in a conservative design. Consider separate channel widths if there are islands, other bridges or channel features in the upstream vicinity of the bridge that would break up or otherwise prevent large unbroken ice floes from impacting the bridge.

14.3.6 Live Load Surcharge (LS)

Reference: LRFD Article 3.11.6.2

When reinforced concrete approach slabs are provided at bridge ends, live load surcharge need not be considered on the end bent; however, the reactions on the end bent due to the axle loads on the approach slabs shall be considered.

It is MDT policy to design the end bents to allow the eventual use of approach slabs but not to initially use them. This allows for an approach slab to be added later. Thus, the end bents must be able to resist the reactions due to axle loads on an approach slab and the lateral pressure due to the live load surcharge but not both in combination.

Design Parameters Used			Information to show in Plans		
Pressure (MPa)	t = 300 (mm)	t = 460 (mm)	Light	Moderate	Severe
0.38	✓		✓		
0.77	✓			✓	
1.15		✓			✓

ICE LOADING PARAMETERS
Figure 14.3B

14.4 ELASTIC STRUCTURAL ANALYSIS

14.4.1 General

Reference: LRFD Article 4.5.2

The LRFD Specifications are a hybrid design code in that the force effects on the load side of the LRFD equation are determined through elastic analysis procedures in most cases, yet the components' resistances are based on inelastic responses. The hybrid nature of structural design is acceptable based on the assumption that the inelastic component of structural performance will always remain relatively small because of redistribution of force effects. This redistribution is assured by providing adequate redundancy and ductility of the structures, which is MDT's general policy for the design of bridges.

Section 14.4 discusses the approximate analysis of girder-slab superstructures and their longitudinal and transverse components. The Section also provides methods of analysis for distribution of lateral loads such as wind and centrifugal force by frame action and/or diaphragms and for axial and flexural effects of imposed deformations such as elastic shortening, creep, shrinkage, temperature and settlement.

14.4.2 Influence Lines

Influence lines can serve as a tool to determine load positions for maximum effect and for evaluating the magnitude of that effect. Constructing an influence line can consist of dividing the structure into eight or ten equal intervals and calculating the force effects due to a unit load at each resulting node.

Recognizing that the influence line is essentially a deflection diagram drawn for a unit relative displacement introduced into the structure at the point of interest can simplify the process. For flexure, consider the relative displacement to represent a unit rotation.

14.4.3 Distribution of Live Load in Multi-Girder Superstructures

Reference: LRFD Article 4.6.2.2.1

14.4.3.1 General

Distribution, for the purpose of this Section in the **MDT Structures Manual**, means the determination of the maximum portion of the total applied live load that may be carried by an individual girder of the superstructure.

14.4.3.2 Load Distribution Factors in the LRFD Specifications

Reference: LRFD Articles 4.6.2.2.1, 4.6.2.2.2 and 4.6.2.2.3

LRFD Article 4.6.2.2.2 presents several common bridge superstructure types, with empirically derived equations for live load distribution factors for each one. Each distribution factor gives a fraction of a lane live load to apply to a girder to evaluate it for moment or for shear. The factors account for interaction among loads from multiple lanes.

The empirical formulas result from regression analyses performed on results of finite element analyses of a large sample of typical superstructures. The equations are intended to produce results within 5% of the finite element analyses on which they rely. See *Distribution of Wheel Loads on Highway Bridges* by T. Zokaie, T. A. Osterkamp and R. A. Imbsen, Final Report, NCHRP Project No. 12-26, for details on the development of the distribution factors.

Distribution factors simplify the design process and minimize potential modeling errors. They reduce the problem of modeling the entire bridge from a two- or three-dimensional analysis down to a one-dimensional analysis of a girder.

Some assumptions that allow this model simplification are:

1. Relative stiffness among different parts of a girder determines longitudinal distribution of live load moment. Deck stiffness determines the transverse distribution. Live load moment distribution factors must include properties of the girder and deck. The calculation of live load moment distribution then becomes an iterative process, in which the designer assumes properties, tests the moment distribution and resulting stresses, then modifies section properties and repeats the process.

2. Force effects, moments and shears that control design consist of the extreme case for each one. Because extreme effects can occur anywhere along the girder, the extreme moment and the extreme shear rarely occur together in the same location or due to the same loading.

3. Distribution factors assume the same vehicle or load in all lanes. This assumption makes analyzing special permit vehicles difficult.

4. The distribution factors represent placing loads in design lanes to generate the extreme effect in a specific girder. The location of design lanes is not related to the location of striped lanes on the bridge. Summing all the distribution factors for all the girders produces a number of design lanes greater than the bridge can carry. This occurs because each girder must be designed for the maximum load it could be subjected to.

14.4.3.3 Refined Analysis in the LRFD Specifications

Reference: LRFD Articles 4.6.2.2 and 4.6.3

The more sophisticated distribution-factor equations are analytically superior to the old "S over" factors which have been used for bridges with spans and girder spacings far beyond those for which they were originally developed.

The tables of distribution factors given in LRFD Article 4.6.2.2 include a column entitled "Range of Applicability." The LRFD Specifications suggests that bridges with parameters falling outside the indicated ranges be designed using the refined analysis requirements of LRFD Article 4.6.3. In fact, these ranges of applicability do not necessarily represent limits of usefulness of the distribution-factor equations, but they represent the range over which bridges were examined to develop the equations. Other states have conducted parametric studies to extend these ranges for typical bridges in their states which have demonstrated that the factors can be used far outside of the range of certain parameters which were specifically studied. Therefore, MDT policy is to use refined analysis only with the approval of the Bridge Design Engineer and only where the simple distribution factors are clearly inadequate. One example may be where the overhang limitations are exceeded.

Refined analysis includes both two- and three-dimensional models. The study that developed the simple distribution factors also investigated refined analysis methods. The study showed that the extra complication of three-dimensional analysis provides no additional value when compared to a more simple two-dimensional grid analysis. Typically, in a grid analysis, longitudinal elements represent the girders including any composite deck, and the transverse elements represent the deck. LRFD Article 4.6.3.3 gives general requirements for grid analysis in terms of numbers of elements and aspect ratios.

14.4.3.4 Continuous Frames

Reference: LRFD Articles C5.7.3.2, 5.8.3.2 and 5.11.1.1

Centerline distances shall be used in the analysis of continuous frame members.

For concrete frames, the value of the moment of inertia for the computation of flexural stiffness of slabs, girders, columns, etc., shall be based on the gross concrete section; the effect of reinforcement may be neglected.

Moments used for designing a section at the support shall be based on the moment value at the face of rectangular columns and at the face of an equivalent square for round columns.

The critical section for bond shall be taken at the same place as for negative bending, and the shear used for computing bond shall be based on the same loading and section as for negative bending. Bond should also be investigated at planes where changes of section or of reinforcement occur and at the point of inflection. The flexural bond stress need not be considered in compression nor in those cases of tension where anchorage bond is less than 0.8 of the permissible.

The critical section for shear shall be the "d" distance from the support, as stated in the LRFD Specifications, except when concentrated loads fall within the "d" distance. In this case, the shear shall be checked at the point load and adequate stirrups provided to that point.

14.4.3.5 Overhang

Reference: LRFD Articles 4.6.2.2.1, 4.6.2.2.2d and 4.6.2.2.3b

For the purpose of live-load distribution, large overhangs (i.e., those requiring refined analysis) are defined as those where the roadway portion of the overhang exceeds 1675 mm for I-shaped steel or concrete girders. Because overhang dimensions are limited in Section 15.4.1.2 of this **Manual**, large overhang distribution considerations do not need to be considered for MDT practice.

For economy of construction, allow for the reuse of girders if the bridge is widened in the future, to reduce the probability of misplacing seemingly identical but actually different girders on the construction site; all prestressed girders are fabricated to the governing condition, interior or exterior. For economy in fabrication, steel girders are also typically fabricated to the governing condition in terms of web-plate and flange-plate sizes and transitions.

14.4.3.6 Number of Girders

Reference: LRFD Article 9.7.2.4

Studies indicate that the cost of a bridge is directly proportional to the number of girders in the cross section. Two-girder arrangements are discouraged because of concerns for the level of redundancy. Article 9.7.2.4 of the LRFD Specifications implies that, with a 200-mm thick non-prestressed concrete deck slab, the girder spacing can safely be extended to approximately 4000 mm. Although not commonly used, a three-girder layout is possible for a bridge site that carries a narrow roadway where span lengths are not at the maximum for the girder type used.

14.4.4 Wind Load Distribution by Frame Action

Reference: LRFD Articles 3.4.1, 3.8.1 and 4.6.2.7.1

Wind load in the LRFD Specifications is addressed in LRFD Article 3.8.1. The basic wind load, acting normal to a surface for a 160-km/h wind velocity at a height not exceeding 10 m above ground level, is 2.4 kN/m^2. Surfaces exposed to the wind should normally include that of the girders, deck, curbs and/or barriers.

In accordance with Article 3.4.1 of the LRFD Specifications, the effects of wind load should be investigated for strength Limit States III and V. For Limit State III, wind load at full value should be considered with a load factor of 1.4. The absence of live load at this limit state reflects that, above approximately 90-km/h wind velocity, vehicles become dynamically unstable and tend to stay off the bridges. For Limit State V, wind load should be assumed to be acting on both structural and vehicular surfaces, with a load factor of 0.40. This factor is actually the product of the 1.40 load factor and the square of the 90/160 wind velocity ratio.

As specified in LRFD Article 4.6.2.7, wind load may be assumed to be distributed in either of the following three ways:

1. Method 1. For typical MDT practice, the web is laterally supported at the respective centers of the deck and the lower flange. In this case, the lower flange is acting as a lateral beam transmitting wind load on the lower half of the outside girder either to intermediate diaphragms or to the bearings.

   Note: Special circumstances may require Method 2 or 3.

2. Method 2. Horizontal wind bracing in the plane of the flange can distribute the wind load among adjacent flanges directly to the bearings. Except for the largest of bridges, horizontal wind bracing is not necessary nor economical.

3. Method 3. The web is acting as a vertical cantilever, framing into the deck and fixed at its centerline. Maximum vertical flexural stresses occur due to this action at the point where the web joins the top flange, and these stresses must be investigated. In concrete girders, these stresses are normally considerably less than the cracking strength of concrete.

   Where diaphragms are used, investigation of vertical flexural stresses is not required.

   In rolled beams, these stresses are also usually small. This type of action, however, should be investigated in welded plate girders only where the transverse (vertical) stiffeners are welded only to the top flange. If the vertical stresses without intermediate diaphragms are within specified limits, none are required; however, vertical stiffeners may be necessary for stability or other reasons during and/or after construction.

   For composite deck-girder construction, the shear connectors or extended stirrups normally have sufficient reserve to resist the small vertical wind moment and, thus, no investigation is required. In case of non-composite construction, this action should not be used.

The following example illustrates one method of calculation of force effects in an intermediate diaphragm of truss construction. As shown in Figure 14.4A, the 96.0-kN lateral force is the reaction of wind load as carried to the diaphragm by the lower flange. The resultant moment with respect to the top chord of the diaphragm is 96 x 1.5 = 144 kN-m. Vertical forces acting on the four girders are computed on the basis that the diaphragm is infinitely stiff. An inertia-like stiffness of the girder is calculated as:

$$I = \sum_{1}^{n} (1)(x^2) \qquad \text{(Equation 14.4.1)}$$

Where:

1 = represents each girder as a unit

x = distance of the girder from the center of the superstructure

n = number of girders

The resistance for any girder is then:

$$S = I \div x \text{ and } V = M \div S \qquad \text{(Equation 14.4.2)}$$

For the given four-girder superstructure:

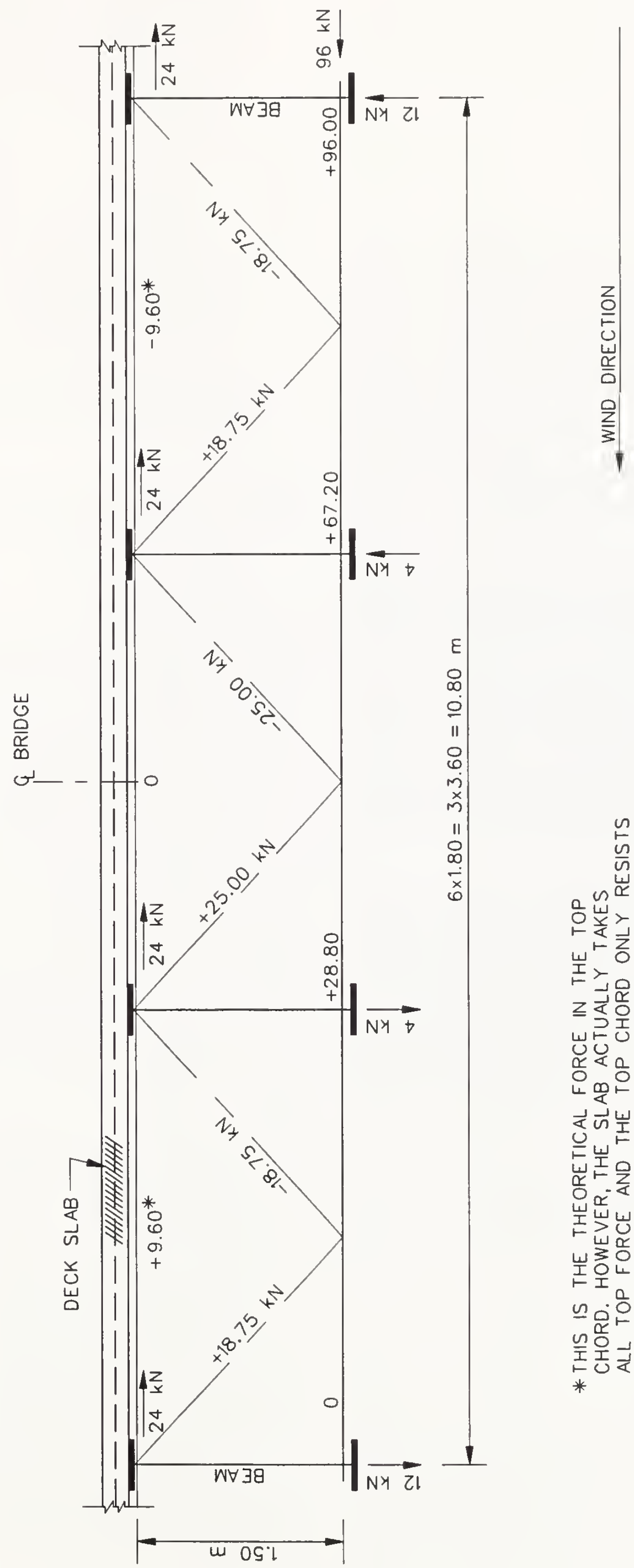
$I = 2[1.8^2 + 5.4^2] = 64.8\ m^2$

For the outside girders:

$S_o = 64.8 \div 5.4 = 12.0$ m
$V_o = 144 \div 12.0 = \pm 12.0$ kN

For the inside girders:

$S_i = 64.8 \div 1.8 = 36.0$ m
$V_i = 144 \div 36.0 = \pm 4.0$ kN



* THIS IS THE THEORETICAL FORCE IN THE TOP CHORD. HOWEVER, THE SLAB ACTUALLY TAKES ALL TOP FORCE AND THE TOP CHORD ONLY RESISTS TRANSVERSE LOADS PRIOR TO CASTING THE DECK.

INTERMEDIATE DIAPHRAGM

Figure 14.4A

Lateral distribution of the wind force among the girders is indeterminate with the outside and inside girders being identical, or close to identical; however, a uniform distribution is acceptable. Accordingly:

$H_o = H_i = 96.0 \div 4 = 24.0$ kN.

Figure 14.4A indicates the calculated truss member forces for the basic wind load in addition to the cross-sectional dimensions of the superstructure. It should be noted for design that wind load is reversible in the diaphragm. Positive member force means compression, negative tension. The top chord is sized to resist construction loads prior to the hardening of the concrete deck, not design wind loads.

Figure 14.4B illustrates the configuration of a truss-like diaphragm required at bearing points. This truss is the reverse version of that selected in Figure 14.4A. The selections are arbitrary in that both systems are acceptable for either application; they are provided for completeness. Furthermore, the system provided in Figure 14.4B can be used for a strut-and-tie analysis of solid concrete diaphragms.

The total wind force is taken as 480 kN. Vertical forces on the girders are calculated the same way as for the intermediate diaphragm. The wind force is uniformly distributed (120 kN) at both the top and bottom of girders. Force effects in truss members are shown in Figure 14.4B.

14.4.5 Superimposed Deformations

Reference: LRFD Article 3.12

14.4.5.1 General

Superimposed deformations have little effect on common typical bridges such as slab-on-girder bridges. On these bridges, only the effects of temperature are typically considered in sizing expansion devices or in considering stresses and deformation in integral-type bridges. Superimposed deformations have special significance for some less common bridges, such as segmentally constructed concrete bridges.

Superimposed deformations include:

1. elastic shortening (ES),
2. creep (CR),
3. shrinkage (SH),
4. temperature (TU and TG), and
5. settlement (SE).

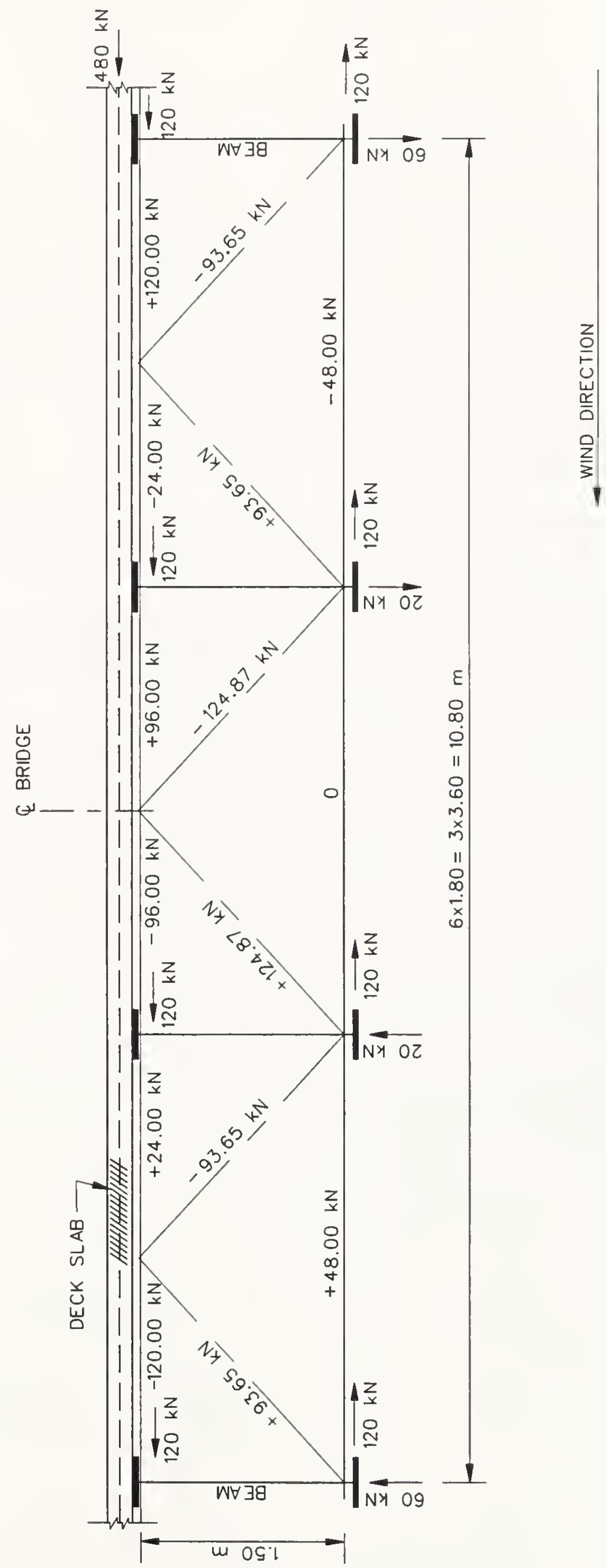
The analysis of force effects due to settlement is provided in Section 14.4.5.2.

14.4.5.2 Force Effects Due to Settlement

Reference: None

Settlement is a downward (positive) movement of a pier or abutment caused by slip, consolidation or failure of the supporting soil. On rare occasions, an upward (negative) displacement may occur. This analysis may be necessary when problems occur on bridges in-service or under construction. They are not needed for routine design. The following discussion assumes uniform settlement transversely for each foundation. Non-uniform settlement or rotation of a foundation adds an extra layer of complexity to the analysis but can be treated in much the same manner by quantifying individual girder seat settlements.

The method of analysis calculates force effects in the superstructure due to settlement of the substructure. Pier displacements "Δ_i" are relative values, normalized with respect to the movements of the outside substructure units, as illustrated in Figure 14.4C. The normalization process in this instance consists of constructing a mathematical "string-line" between the extreme ends of the bridge and calculating the deviation of the interior supports from this "string-line." The actual and normalized settlements are presented in the following example:



DIAPHRAGM AT BEARINGS

Figure 14.4B

	Actual	Normalized
Pier 1	0	0
Pier 2	+7.0	+6.2
Pier 3	0	-2.6
Pier 4	+4.0	0

The following equation is a useful tool for determining the moments caused in a continuous structure due to settlement of one or more supports. It is based on the equation for deflection of a simply supported beam of constant inertia, and the equation is found (or its variation) in many engineering textbooks:

$$d_{ij} = \frac{Pc^3}{6EI}\left[1-\alpha^2-\beta^2\right]$$

where:

d_{ij} = deflection at point 'i' due to a load at point 'j'

P = concentrated load

c = total length of beam

EI = rigidity of beam

α = x/c, where 'x' is the distance of point 'i' from the end support

β = b/c where 'b' is the distance of point 'j' from the end support

For further discussion, simplify the above to:

$$k_{ij} = \alpha\beta\left[1-\alpha^2-\beta^2\right]$$

k_{ij} values are calculated as follows:

1. α = 0.20 β = 0.80
   k_{22} = (0.20)(0.80) [1 - 0.20^2 - 0.80^2]
   = 0.0512

2. α = 0.20 β = 0.35
   k_{23} = (0.20)(0.35) [1 - 0.20^2 - 0.35^2]
   = 0.0586

3. α = 0.35 β = 0.20
   k_{32} = (0.20)(0.35) [1 - 0.20^2 - 0.35^2]
   = 0.0586

4. α = 0.35 β = 0.65

   k_{33} = (0.35)(0.65) [1 - 0.35^2 - 0.65^2]
   = 0.1035

Because:

$$k_{22}P_2 + k_{23}P_3 = \frac{6EI}{C^3}\Delta_2$$

and

$$k_{32}P_2 + k_{33}P_3 = \frac{6EI}{C^3}\Delta_3$$

one can insert the variables and solve for reactions, then moments:

$$+0.0512\ P_2 + 0.0586\ P_3 = +6.2\frac{6EI}{C^3}$$

$$+0.0586\ P_2 + 0.1035\ P_3 = -2.6\frac{6EI}{C^3}$$

from which: $P_2 = +426.2\dfrac{6EI}{C^3}$

$P_3 = -266.4\dfrac{6EI}{C^3}$

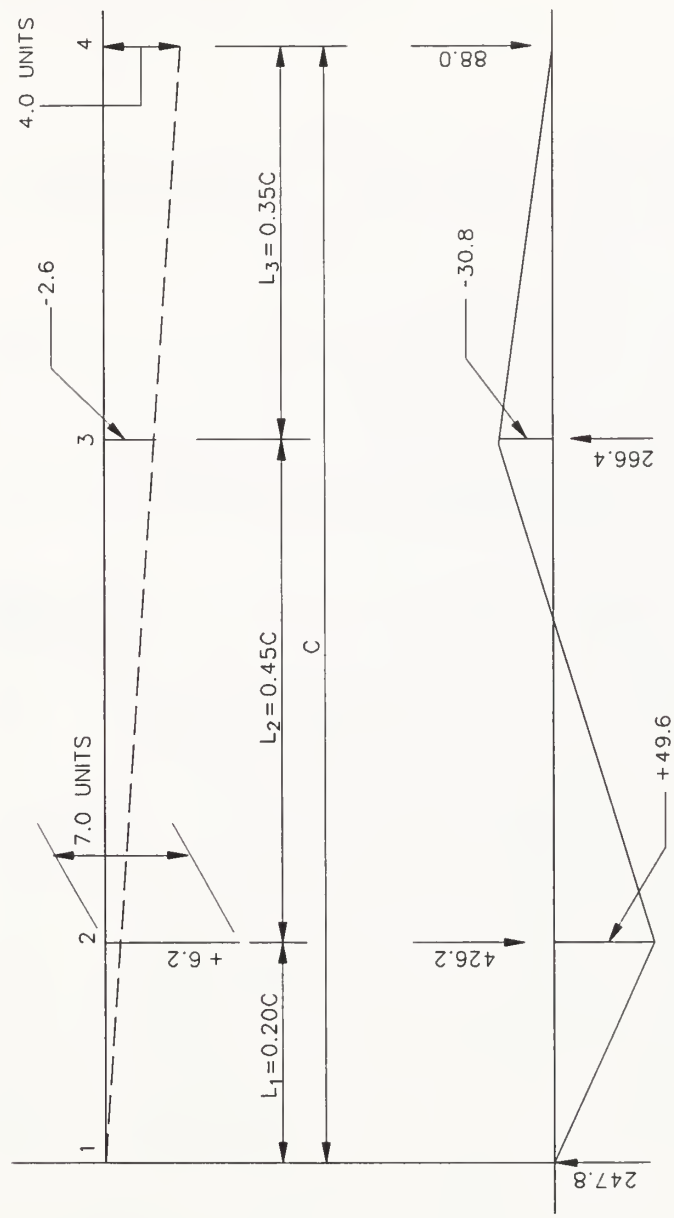
then: $P_1 = -247.8\dfrac{6EI}{C^3}$

$P_4 = +88.0\dfrac{6EI}{C^3}$

$M_2 = +49.6\dfrac{6EI}{C^2}$

$M_3 = -30.8\dfrac{6EI}{C^2}$

Forces and moments are indicated in Figure 14.4C.



FORCE EFFECTS DUE TO SETTLEMENT

Figure 14.4C

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Chapter Fifteen
BRIDGE DECKS

Sections 3, 4 and 9 of the **LRFD Bridge Design Specifications** present the AASHTO criteria for the structural design of bridge decks. Section 3 specifies loads for bridge decks, Section 4 specifies their analyses and Section 9 specifies the resistance of bridge decks. Unless noted otherwise in Chapter Fifteen of the **Montana Structures Manual,** the LRFD Specifications apply to the design of bridge decks in Montana. Chapter Fifteen presents information on MDT practices in the design of bridge decks.

15.1 BACKGROUND

15.1.1 Bridge Decks and Superstructures

The LRFD Specifications encourage the integration of the deck with the primary components of the superstructure by either composite or monolithic action. In some cases, the deck alone is the superstructure. The LRFD Specifications call this a slab superstructure; MDT calls it a flat slab. More commonly, the deck in conjunction with its supporting components comprises the superstructure, leading to some confusion in definition.

Chapter Fifteen of the **Montana Structures Manual** documents MDT criteria on the design of bridge decks which are constructed in conjunction with prestressed, precast concrete I-beams or composite steel I-beams. Chapter Sixteen discusses the design of reinforced, CIP concrete slabs.

Also note that the LRFD Specifications have introduced the "empirical" deck design. Decks designed using the empirical design method are also sometimes called isotropically reinforced or Ontario-type decks. MDT is evaluating this design for potential future use in Montana.

15.1.2 Durability of Concrete Bridge Decks

Reference: Various LRFD Articles

As stated in the commentary to LRFD Article 2.5.2.1.1, the single most prevalent bridge maintenance problem is the deterioration of concrete bridge decks. Measures to enhance the durability of concrete components, in particular, are discussed in Article 5.12.

The distress of bridge decks, and their premature replacement, has become a serious problem in the United States. In Article 1.2, the LRFD Specifications define the design life of new bridges to be 75 years. Thus, designers are compelled to re-evaluate conventional wisdom regarding the long-term performance of concrete bridge decks.

15.1.3 Protection of Reinforcing Bars

In the presence of air and moisture, reinforcing steel corrodes, and the corrosion process is accelerated by salts. The corrosion product (i.e., rust) has a larger volume than the steel consumed, resulting in spalled areas at the top of the deck.

There are a variety of methods to protect the reinforcing steel on new decks and to decelerate the rate of corrosion as identified below; however, Montana typical practice is to use epoxy-coated reinforcing steel in both mats of deck reinforcing combined with a minimum cover of 60 mm from the top surface of the deck to the top mat:

1. Epoxy-Coated, Galvanized, Stainless Steel/ Stainless Steel-Clad Reinforcing Steel. Retards corrosion of reinforcing steel.

2. Waterproofing and Asphaltic Overlay. Experience indicates that waterproofing cannot be made perfect and, by potentially trapping moisture, it may be counter-productive. MDT does not permit its usage except in special cases with advance approval of the Bridge Engineer.

3. Concrete Overlay (Latex-Modified). Because they are extremely impervious, they perform well on old decks but, because they are expensive, they are not always cost effective.

4. Additional Cement. This is an effective way of reducing permeability but, because of increasing shrinkage, can be counter-productive. MDT is monitoring nationwide developments in high-performance concrete (HPC) to evaluate its potential use in Montana.

5. Fly Ash. This moderately decreases permeability, contributes to a gain in strength and improves concrete durability.

6. Microsilica. This is an effective internal sealant but produces high hydration temperatures and plastic cracking.

7. Calcium Nitrate. This absorbs chloride by sacrificial chemical binding and, thus, delays but does not prevent migration of chloride ions.

8. Concrete Cover. The practical depth of cover delays but does not prevent the chloride from reaching the steel bars.

9. Surface Sealants. Provides initial protection of the deck.

10. Transverse Post-Tensioning. Minimizes cracking.

11. Cathodic Protection. Retards corrosion of reinforcing steel.

12. Bar Size. Smaller diameter bars for the same steel cross sectional area provide better crack-size control.

15.2 "STRIP METHOD"

15.2.1 Application of the "Strip Method" to Composite Concrete Decks

Reference: Appendix to LRFD Section 4

The application of the strip method to composite concrete decks is represented by a design aid in the Appendix to Section 4 of the LRFD Specifications, Table A4.1-1. An introduction to the Table discusses the limitations on its application.

In lieu of a rigorous analysis, Figure 15.2A may be used to design the concrete deck reinforcement. Figure 15.2A tabulates the results of slab steel design traditionally used by MDT for Grade 420 reinforcing steel and concrete strengths of 28 MPa and 31 MPa. Because the calculations do not show a particular sensitivity to concrete strength, Figure 15.2A is appropriate for both concrete strengths. Figure 15.2A is based upon the AASHTO **Standard Specifications for Highway Bridges.** Rigorous application of the strip method generally results in slightly greater reinforcement requirements than presented in the Figure. Based upon satisfactory past performance and the fact that the "empirical" deck design method of the LRFD Specifications require less reinforcement than the "strip method," designs in accordance with Figure 15.2A are deemed acceptable.

Slab steel design as presented in Figure 15.2A attempts to balance the costs of concrete and reinforcing steel to produce an optimum design.

15.2.2 Typical Reinforcement

Figure 15.2B presents the typical deck reinforcement designed in accordance with Figure 15.2A.

28 MPa or 31 MPa Concrete

SLAB STEEL DESIGN*									
MAXIMUM BEAM SPACING (mm)					EFFECTIVE SPAN (mm)	SLAB T (mm)	TRANSVERSE STEEL (mm)	#13 DIST. STEEL	
I	A	IV	M72	10A				0.50 S	0.25 S
1676	1778	1880	1981	1969	1375	185	#16 @ 230	4	1
1753	1854	1956	2057	2045	1450	185	#16 @ 225	4	1
1803	1905	2007	2108	2096	1500	185	#16 @ 220	4	1
1854	1956	2057	2159	2146	1550	190	#16 @ 220	4	1
1930	2032	2134	2235	2223	1625	190	#16 @ 215	4	1
2007	2108	2210	2311	2299	1700	190	#16 @ 210	5	2
2057	2159	2261	2362	2350	1750	190	#16 @ 205	5	2
2134	2235	2337	2438	2426	1825	190	#16 @ 195	5	2
2184	2286	2388	2489	2477	1875	200	#16 @ 205	5	2
2235	2337	2438	2540	2527	1925	200	#16 @ 205	5	2
2311	2413	2515	2616	2604	2000	200	#16 @ 200	6	2
2388	2489	2591	2692	2680	2075	200	#16 @ 190	6	2
2464	2565	2667	2769	2756	2150	200	#16 @ 185	7	2
2565	2667	2769	2870	2858	2250	200	#16 @ 180	7	2
2692	2794	2896	2997	2985	2400	205	#16 @ 175	8	2
2794	2896	2997	3099	3086	2500	205	#16 @ 170	8	2
2870	2972	3073	3175	3162	2575	205	**#16 @ 165	9	3
2921	3023	3124	3226	3213	2625	205	**#16 @ 160	9	3
3023	3124	3226	3327	3315	2725	210	**#16 @ 160	9	3
3124	3226	3327	3429	3416	2825	210	**#16 @ 155	10	3
3200	3302	3404	3505	3493	2900	210	**#16 @ 150	11	3
					3000	210	#19 @ 195	12	3
					3100	210	#19 @ 190	13	4
					3225	210	#19 @ 185	13	4
					3375	215	#19 @ 185	14	4
					3425	220	#19 @ 185	14	4
					3525	220	#19 @ 180	15	4
					3600	225	#19 @ 180	15	4
					3650	230	#19 @185	15	4
					3800	230	#19 @180	16	4

Minimum haunch = 220 mm (Standard) (rows 1676 through 2565)

Min. haunch = Slab + 20 mm (rows 2692 through 3200)

* This table is for ASTM A-615 Grade 60 steel and 28 MPa or 31 MPa concrete <u>only</u>.

**B2 barrier bars are #13.

Notes:

1. Check haunch depth on steel structure and on sag verticals.
2. Use same haunch depth for all spans.

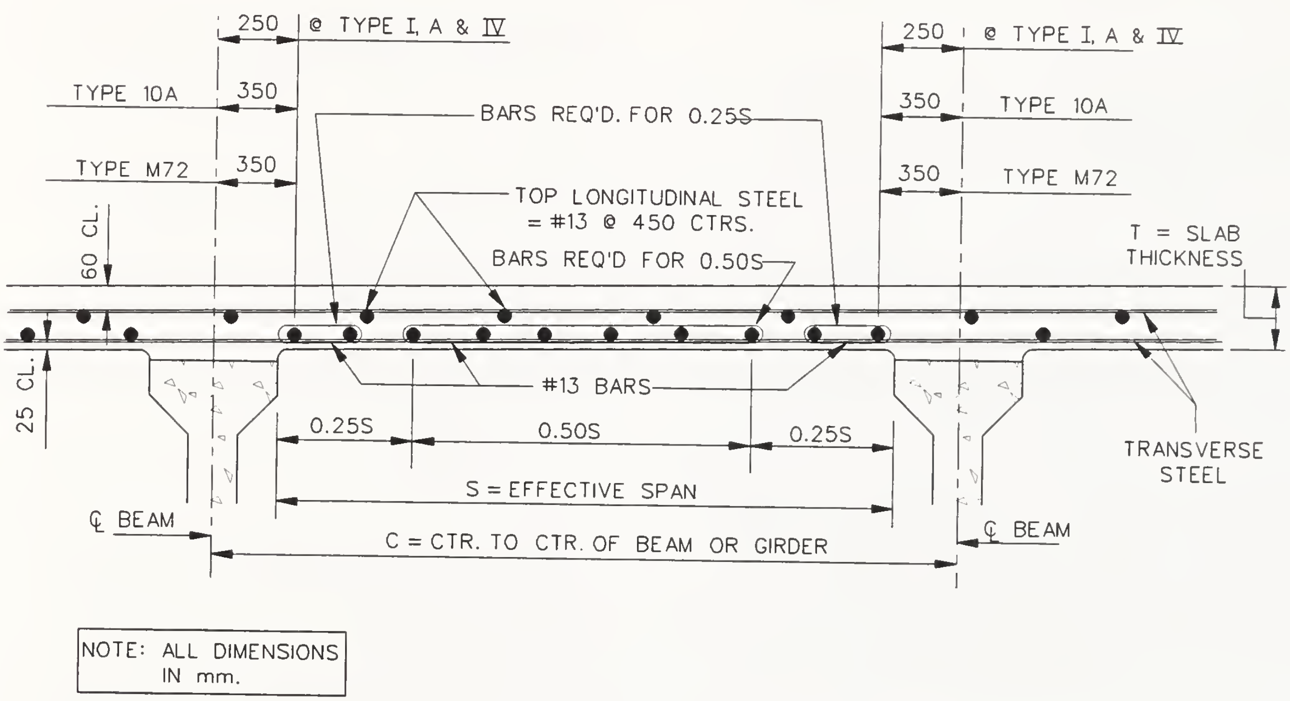
<u>EFFECTIVE SPAN</u>

Steel Griders:	S = C – ½ Flange Width
Type I:	S = C – (304) Flange Width
Type A:	S = C – (406) Flange Width
Type IV:	S = C – (508) Flange Width
Type M72:	S = C – (610) (4) (Flange Thickness)
Type 10A:	S = C – (597) (Flange + Web) / 2

$$\text{Minimum Slab Thickness } (T) = \frac{(S + 3048)}{30} \text{ mm, but not less than 165 mm}$$

SLAB STEEL DESIGN
(Strip Method)

Figure 15.2A



STRIP METHOD DESIGN
(Typical Deck Reinforcement)

Figure 15.2B

15.3 DESIGN DETAILS FOR BRIDGE DECKS

15.3.1 General

The following general criteria shall apply to bridge deck design:

1. Thickness. The depth of reinforced concrete decks shall not be less than 165 mm.

2. Reinforcement Steel Strength. The specified yield strength of reinforcing steel shall not be less than 420 MPa.

3. Reinforcement Cover. The bottom reinforcement cover shall be a minimum of 25 mm. The top reinforcement cover shall be a minimum of 60 mm. The primary reinforcement shall be on the outside.

4. Rebar Spacing. The minimum spacing is 150 mm. The maximum spacing is 450 mm.

5. Minimum Rebar Size. This shall be #13.

6. Sacrificial Wearing Surface. The top 35 mm of the bridge deck shall be considered sacrificial. Its weight must be included as a dead load but its structural contribution to the structure shall not be included in the structural design or as part of the composite section except for deflection calculations.

7. Concrete Strength. The specified 28-day compressive strength of concrete for bridge decks shall be 28 MPa for Districts 3, 4 and 5, unless the District agrees to 31 MPa. This decision will be documented during the preliminary field review. In Districts 1 and 2, 31 MPa strength shall be specified.

8. Epoxy Coating. Epoxy coating shall be used for all reinforcing steel in both top and bottom layers in bridge decks.

9. Length of Reinforcement Steel. The maximum length of reinforcing steel in the deck shall be 12.19 m for #13 bars and 18.29 m for larger diameter bars.

10. Placement of Transverse Reinforcing on Skewed Bridges. The following applies:

    a. Skews < 15°: Place the transverse reinforcing steel parallel to the skew. Even at skews of less than 15°, attempting to place transverse reinforcement on the skew may conflict with shear hoops on prestressed girders. Investigate and do not place transverse steel on the skew if it conflicts.

    b. Skews > 15°: Place the transverse reinforcing steel perpendicular to the centerline of roadway or the long chord of the structure on curved bridges.

    See Section 15.3.4 for structural considerations related to skewed reinforcing placement.

11. Dead Load. For contingencies, add 0.50 kN/m^2 to the dead load weight.

12. Splices/Connectors. Use lap splices for deck reinforcement unless special circumstances exist. Mechanical connectors may be used where clearance problems exist or on a phase construction project that precludes the use of lap splices.

15.3.2 Dimensional Requirements for Concrete Decks

15.3.2.1 Slab Thickness

Figure 15.2A presents standard deck slab thicknesses and required slab reinforcement for given beam spacings and Grade 420 reinforcing steel.

15.3.2.2 Haunch Dimensions for Steel Girders

Figures 15.3A and 15.3B illustrate the controlling factors used to determine the haunch dimensions for steel girders.

The slab thickness is selected based on Figure 15.2A or the project-specific slab design.

The first check is to determine the minimum "D" dimension that places the shear studs underneath the top mat of reinforcing steel.

The second check determines the minimum "D" dimension that allows the transverse bottom reinforcement to clear the top flange of the girder and provide 25 mm of clearance to the bottom of the slab.

The greater value of these two checks for the minimum "D" dimension controls. A value of "D" should be selected to the next higher 10 mm increment:

1. Control dimension "D" should be established immediately after the top flange plate and after the splice plate thicknesses have been determined. In real world context, preliminary beam runs are usually sufficient, and it is not necessary to re-examine the issue after the final beam runs are made.

2. Control dimension "D" should be held constant for all plate girders throughout the structure. For rolled girders, this dimension may vary along the span.

3. Once established, control dimension "D" should be used for all elevation computations such as bridge seats, top of splice elevations, etc.

15.3.2.3 Haunch Dimensions for Concrete Beams

The haunch dimension for concrete beams is the distance between the top of the deck and the top of the beam and occurs at the top flange corner on the low side of the sloping crown or superelevation as shown in Figure 15.3C. This dimension varies along the length of a concrete beam due to profile grade, beam camber and dead load deflection. The haunch dimension shown on the plans is the minimum haunch at centerline bearing at the side of the beam where the slab elevation is the lowest and is used more for design purposes than in construction of the deck.

Dimension "B" is the distance between the top of beam and the bottom of the deck and also varies along the length of a concrete beam. Due to roadway profile grade and unpredictable beam camber, this distance should be 20 mm or greater at all locations within the span. This will provide room for placement of the chamfer strip during construction, and it will reduce the possibility that the shear steel (stirrups) and/or the top of the beam will interfere with the slab reinforcing steel.

Dimension "A" is at the centerline of the girder and is used in prestressed beam design computer programs. It is calculated using the following equation:

$$A = B + \left(\frac{\text{Top Flange Width}}{2}\right)(\%\ \text{Crown Slope})$$

(Equation 15.3.1)

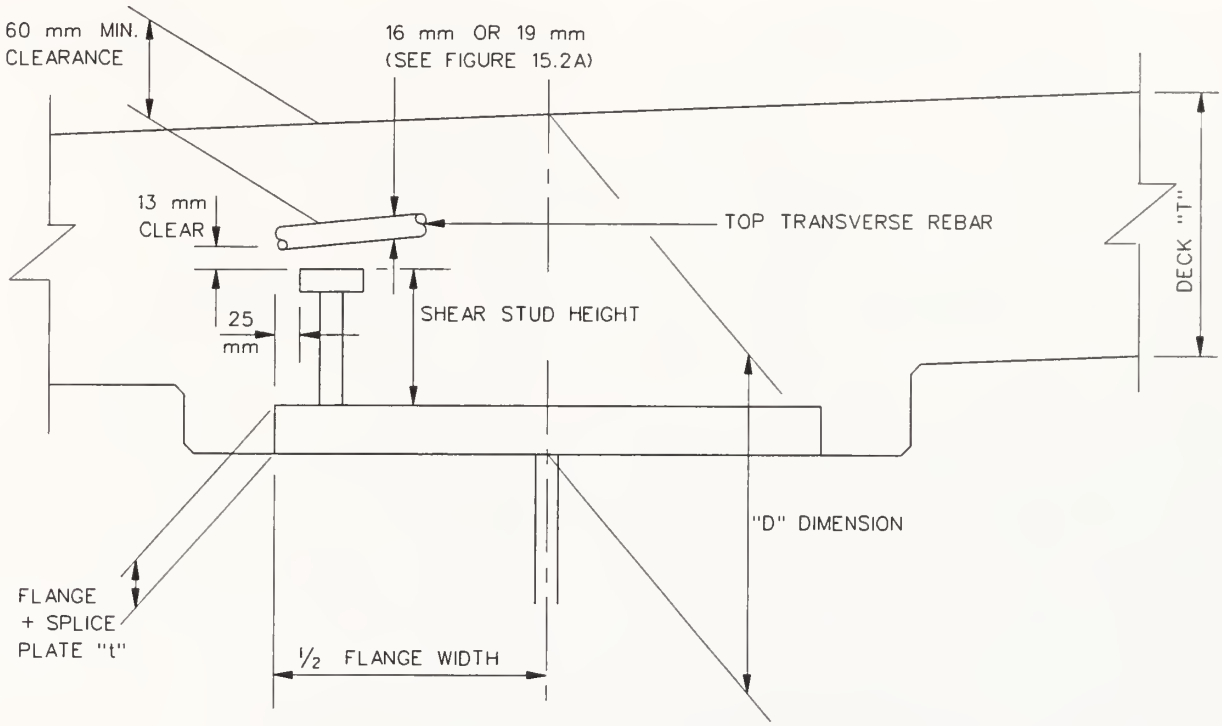
15.3.3 Forms

Stay-in-place forms are not allowed. Removable forms are used to support deck overhangs and decks between girders in any type of structure. This allows a visual inspection of the deck underside.

15.3.4 Skewed Decks

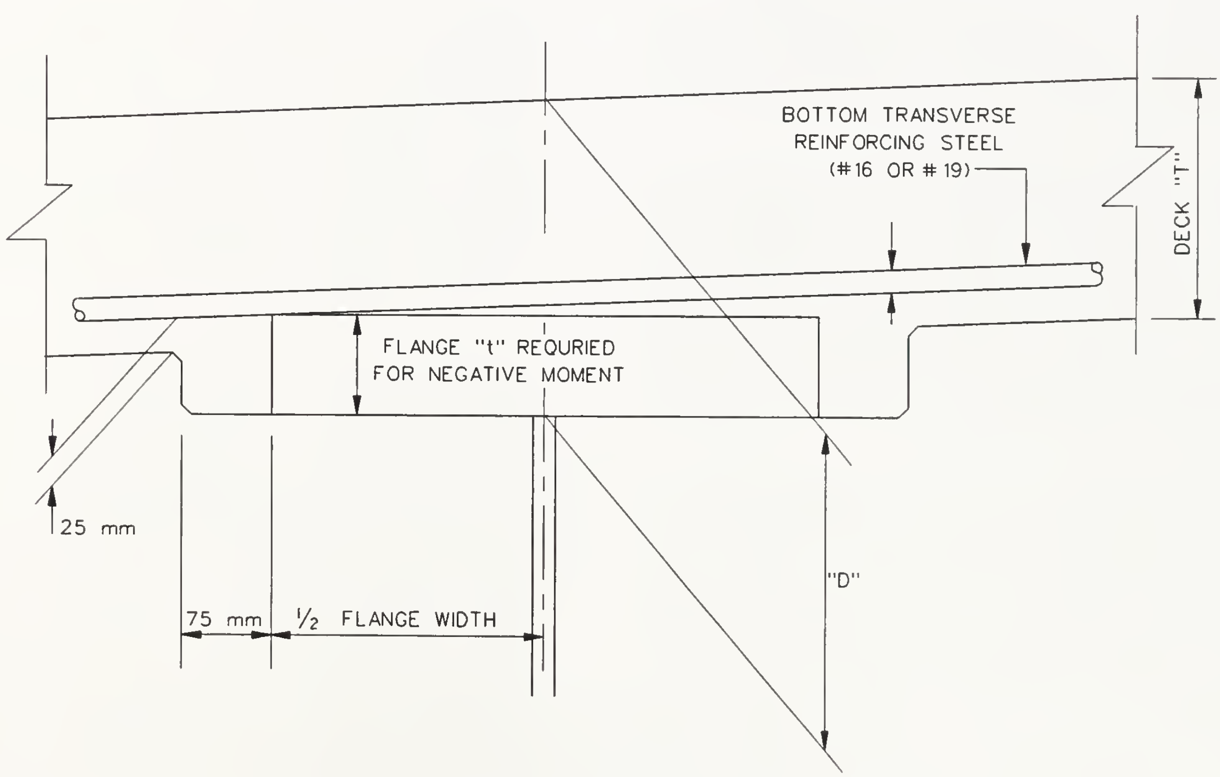
Reference: LRFD Article 9.7.1.3

Skew is defined by the angle between the end line of the deck and the normal drawn to the longitudinal centerline of the bridge at that point. The two end skews can be different. MDT practice is that the maximum skew angle on a bridge without approval is 35°. The Bridge Area Engineer must approve the use of greater skew angles. Also, the bridge skew should not



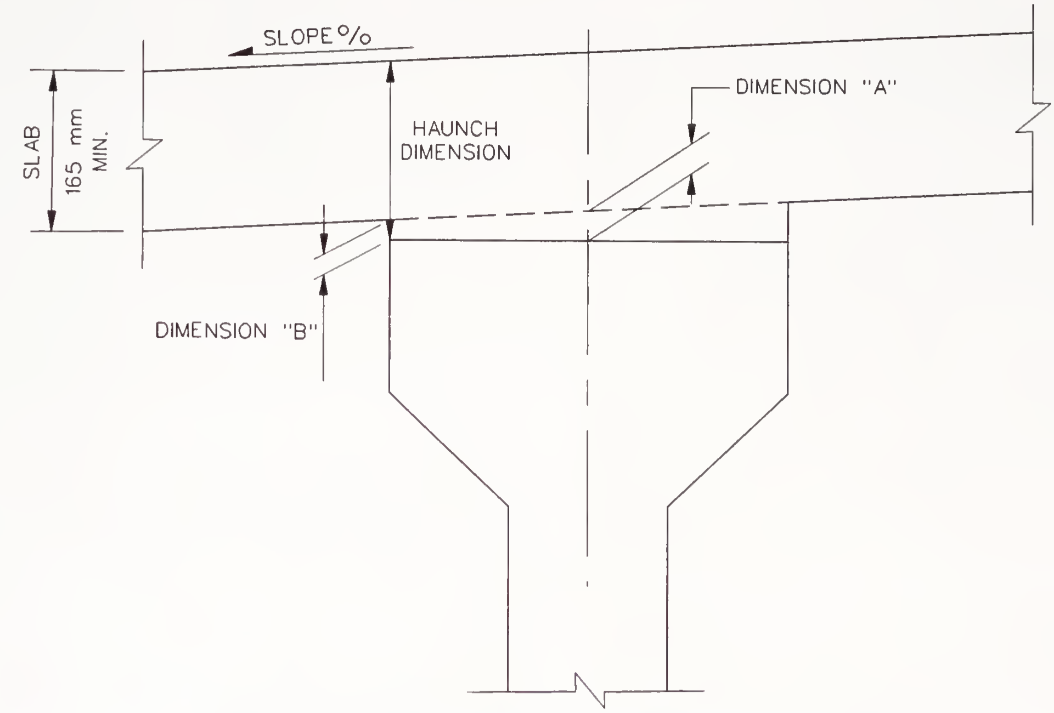
HAUNCH DIMENSIONS FOR STEEL BEAMS

Figure 15.3A



HAUNCH DIMENSIONS FOR STEEL BEAMS

Figure 15.3B



HAUNCH DIMENSIONS FOR CONCRETE BEAMS

Figure 15.3C

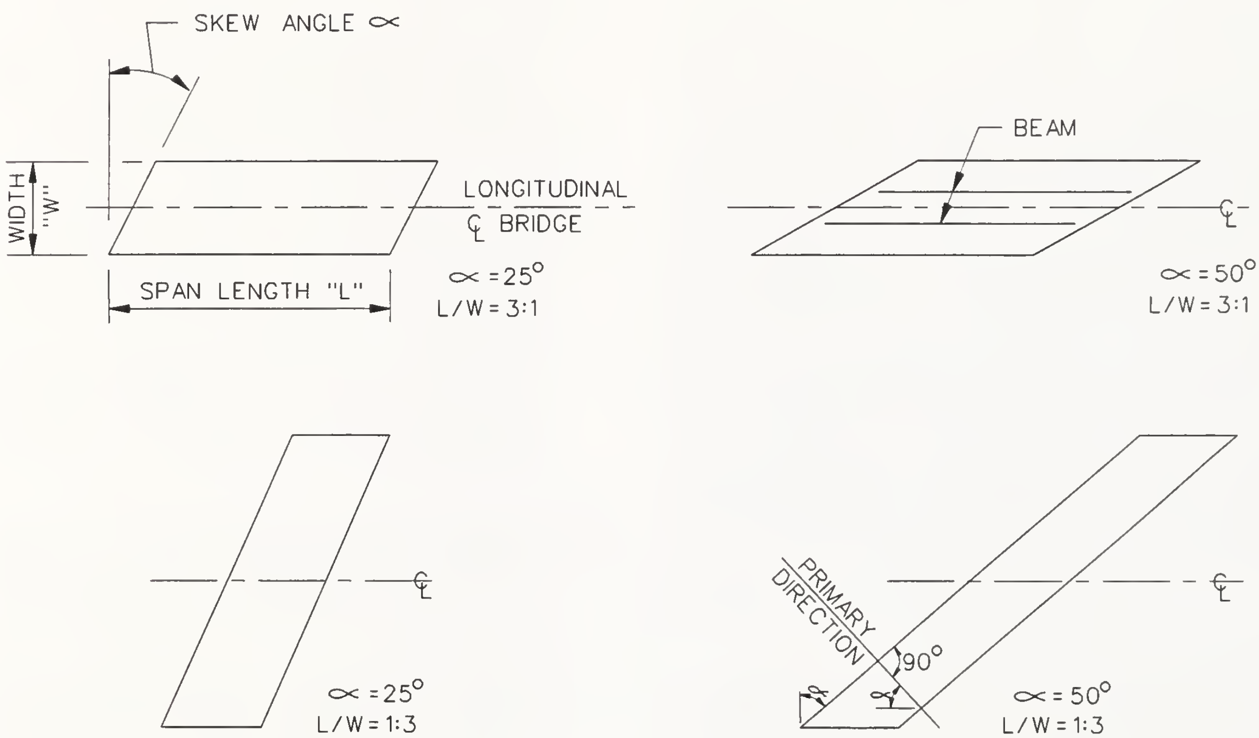
match the angle of a snowplow, which is 35° to 37° right. In addition to skew, the behavior of the superstructure is also affected by the span-length-to-bridge-width ratio.

The LRFD Specifications generally imply that the effects of skew angles not exceeding 25° can be neglected for concrete decks, but the LRFD Specifications assume the typical case of bridges with relatively large span-length-to-bridge-width ratios. Figure 15.3D illustrates four combinations of skew angles 25° and 50° and length-width ratios of 3:1 and 1:3.

Both the 50° skew and the 1:3 length-width ratio are considered extreme values for bridges, but this often occurs where the deck constitutes the top slab of a culvert. It can be judged visually that both combinations with 25° skew may be orthogonally modeled for design with the skew ignored.

The Commentary to Section 9 of the LRFD Specifications provides valid arguments supporting the limit of 25° concerning the direction of transverse reinforcement. It suggests that running the transverse reinforcement parallel to a skew larger than 25° will create a structurally undesirable situation in which the deck is essentially unreinforced in the direction of principal stresses. It is required that, for skew > 25°, the transverse reinforcement must be laid perpendicular to the beams.

The combination of 50° skew and L/W = 1:3, as indicated in Figure 15.3D, produces a peculiar layout. If the deck is a cast-in-place concrete slab without beams, the primary direction of structural action is one being perpendicular to the end line of the deck. Because of the geometry of the layout, consideration should be given to running the primary reinforcement in that direction and fanning it as appropriate in the side zone. With that arrangement, the secondary reinforcement could then be run parallel to the skew, thus regaining the orthogonality of the reinforcement as appropriate for this layout.



COMBINATION OF SKEW ANGLE AND SPAN LENGTH/BRIDGE WIDTH RATIOS

Figure 15.3D

15.3.5 Deck Joints

This Section discusses longitudinal open joints and deck construction joints in decks supported by girders.

15.3.5.1 Longitudinal Open Joints

Reference: LRFD Article 14.5.1.1

Longitudinal open joints are not required in concrete bridge decks with widths of 27 m or less. For deck widths wider than 27 m, a longitudinal open joint may be used or a longitudinal closure pour, not less than 0.60 m wide, may be employed. Transverse steel lap splices shall be located within the longitudinal closure pour. Such a joint should remain open as long as the construction schedule permits to allow transverse shrinkage of the deck concrete. The designer should consider the deflections of each side of the bridge on either side of the closure pour to ensure proper transverse fit up.

15.3.5.2 Construction Joints

Construction joints create planes of weakness that frequently cause maintenance problems. In general, deck construction joints are discouraged and their number should be minimized.

15.3.5.2.1 Longitudinal Construction Joints

The following will apply to longitudinal construction joints:

1. Usage. Construction joints need not be used on decks having a constant cross section where the width is less than or equal to 20 m. For deck widths greater than 20 m (i.e., where the screeding machine span width must exceed 20 m), the designer shall make provisions to permit placing the deck in practical widths.

2. Location. If a construction joint is necessary, do not locate it underneath a wheel line. Preferably, a construction joint should be located over a girder line.

15.3.5.2.2 Transverse Construction Joints

The following will apply to transverse construction joints:

1. Steel Girder Structures. Concrete should be placed continuously on steel girder structures with decks requiring up to a maximum of 125 m^3 of concrete.

   For longer structures that exceed the pour volume limitation of 125 m^3, a slab pouring sequence should be considered in which the deck length is subdivided into segments at the points of final dead load contraflexure, with segments in positive flexure placed first and those in negative flexure last. See also Section 15.3.6.

2. Prestressed Concrete Structures. Prestressed concrete girder bridges made continuous only for live load shall be treated as a special case. Transverse construction joints located 750 mm on each side of the pier centerline shall be provided. The short deck segment and diaphragm over the support provide continuity for live load in the superstructure after the previously poured center regions of the deck have been poured as simple span loads.

3. Location. Where used, transverse construction joints should be placed parallel to the transverse reinforcing steel.

4. Diaphragms. For prestressed concrete girder bridges with cast-in-place decks, the LRFD Specifications require concrete diaphragms at the bearings.

5. Steel Structures. Place a transverse construction joint in the end span of bridge decks on steel superstructures where uplift is a possibility during the deck pour. The condition most likely to produce that form of uplift is a bridge with an end span

relatively short (60% or less) when compared to the adjacent interior span. Uplift during deck pour can also occur at end supports of curved decks and in superstructures with severe skews. If analysis shows that uplift might occur during a deck placement, require a construction joint in the end span and require placing a portion of the deck first to act as a counterweight.

6. End Supports. Live load in other spans can produce uplift in short spans of continuous bridges. If analysis shows this condition may occur, include a counterweight or hold-down devices to counteract the effect. Show the details of these measures on the plans.

15.3.6 Deck Pours

Reference: LRFD Article 2.5.3

The need for a slab pouring sequence in the bridge plans will be based on the volume of concrete in the bridge deck as follows:

1. Less than 75 m^3 — not needed.
2. 75 m^3 to 125 m^3 — case-by-case decision.
3. Greater than 125 m^3 — required.

Where required, the bridge designer shall present in the bridge plans the sequence of placing concrete in various sections (separated by transverse construction joints) of deck slabs on continuous spans. The designated sequence shall avoid or minimize the dead load tensile stresses in the slab during concrete setting to minimize cracking, and it shall be arranged to cause the least disturbance to the portions placed previously. In addition, for longer span steel-girder bridges, the pouring sequence can lock-in stresses far different than those associated with the instantaneous placement typically assumed in design. Therefore, in these bridges, the designer should consider the pouring sequence in the design of the girders.

Figure 15.3E illustrates a sample pour sequence diagram. Also, see Section 5.4.6.4 for guidance on the presentation of the slab pouring sequence detail.

15.3.7 Expansion Joints

Reference: LRFD Articles 14.4 and 14.5

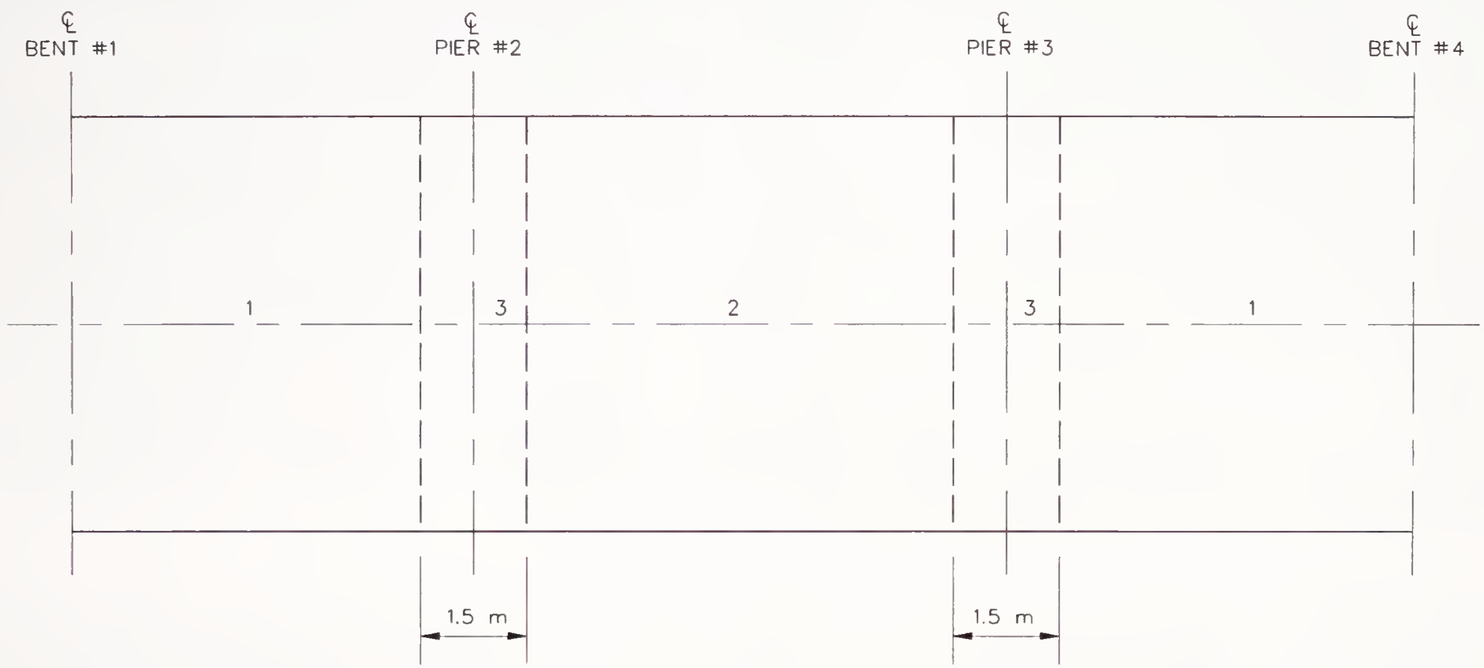
Article 14.4 of the LRFD Specifications provides a discussion on the movements and loads on bridge joints, and Article 14.5 provides in-depth requirements for joints and considerations for specific joint types. The following presents MDT criteria for the use of expansion joints in bridge decks. Expansion length equals the distance from the expansion joint to the point of assumed zero movement.

15.3.7.1 General

Expansion joints in bridge decks are often necessary to accommodate the expansion and contraction of bridges due to temperature variations. The following general criteria apply to all expansion joints in bridge decks:

1. Minimum Length. Because of their inherent operational and maintenance problems, the desirable objective is to eliminate the need for expansion joints. Typically, a reasonably square bridge on flexible bents up to approximately 75 m in length may be constructed without expansion joints.

2. Maintenance Problems. Historically, most of the maintenance problems on bridges in Montana result from failed joints. When a joint fails, this allows debris to fall on top of the bottom flanges and to accumulate on the tops of caps around shoes. This debris is frequently contaminated with chloride containing material and frequently remains moist for extended periods of time. Therefore, the proper selection and design of the expansion joint is a critical design issue. See the remainder of Section 15.3.7 for MDT guidance on selecting the type of expansion joint.



The following note, revised as necessary, will be shown on the plans for continuous prestressed concrete I-beam structures in which the composite slab over the interior supports is designed for the live load:

POUR NUMBERS INDICATE SEQUENCE OF POURS. POURS OVER INTERIOR SUPPORTS WILL BE MADE LAST TO REDUCE THE EFFECT OF THE SLAB DEAD LOAD IN THE NEGATIVE MOMENT AREA. POUR #3 WILL INCLUDE THE DIAPHRAGM AT THE SUPPORT AND WILL BE HELD TO A 1.5–M LENGTH. INTERIOR DIAPHRAGMS WILL BE POURED BEFORE SLAB IS POURED.

TYPICAL POUR DIAGRAM
(Continuous Prestressed Concrete I-Beams)

Figure 15.3E

3. Temperature. Expansion joints shall be designed to accommodate a temperature range of -40°C to 45°C.

4. Angles. If an angle or extruded shape larger than 75 mm x 75 mm will be used to support an expansion joint, the angle must be supported from the top of the beam. Include a detail of the supporting device in the plans.

15.3.7.2 Asphaltic Plug

This joint system is a smooth, durable, load-bearing surface using a combination of polymer-modified asphaltic binder and selected aggregate providing movement ranges up to 50 mm. Its advantages include no mechanical anchorage system, ease of placement, low maintenance and rideability. Its disadvantage include its non-flexibility in cold temperatures and its tendency to rut under heavy traffic in hot temperatures.

15.3.7.3 Silicone Rubber Sealant

The silicone rubber sealant system can be used in joints that have movements up to 50 mm. The movement capacity of this type of joint is dictated by the joint width at the time of installation. The movement capacity is a function of the installation width plus and minus some percent of original gap size. One commonly used product, Dow Corning Product 902 RCS, recommends 100% for maximum opening and 50% for closing movement range; another production uses 50% for opening and closing. This type of joint is maintenance friendly in that local joint failures are easily mended. This system can be bonded to concrete, steel or polymeric elastic cement.

15.3.7.4 Strip Seal

The strip seal expansion joint is the preferred deck expansion joint system for joint movements from 25 mm to 125 mm. Apply the following provisions when sizing a strip seal expansion joint:

1. Size expansion joints for each joint location within the bridge based on the calculated total joint movement. Joint movements are a function of:

   a. girder materials and the coefficient of thermal expansion,

   b. ambient temperature range of bridge location,

   c. expansion length between points of fixity and expansion, and

   d. longitudinal stiffness considerations of substructure elements.

2. Select a joint from the manufacturer's information that provides the required range of movement for the joint being considered. To account for possible improper installation and uncertainty of estimated movements, LRFD Table 3.4.1-1 includes a load factor of 1.20 for the calculated movements due to uniform temperature, shrinkage and creep. With larger movements, this may be difficult to achieve with a strip seal because of the limited sizes of acceptable joint seals.

3. Provide the following plan details and special provisions that identify the specific joint requirements to the contractor:

   a. Show the minimum gap width at the maximum temperature of the design range.

   b. Show a table of temperature versus gap width for various temperatures within the design range.

   c. Show the gap width at the mean temperature.

   d. Provide a factor indicating the temperature change for 3 mm of joint movement.

e. Check the opening at the anticipated installation temperature to see if it meets the manufacturer's requirements.

15.3.7.5 Finger Plates

This joint is applicable to anticipated movements greater than 125 mm. Typically, finger plates are only used on decks supported by steel girders.

Finger plates allow debris to pass through; therefore, a collector trough is required underneath to catch the debris. Almost every collector trough detail is a high-maintenance item with marginal effectiveness. An alternative is to design the finger plate to simply spill all debris through and prepare details at the shoes so that the debris will not cause any adverse effects. However, despite its problems, a well-designed finger plate is perhaps the best design for large-movement joints.

15.3.7.6 Modular

The LRFD Specifications recognizes that modular seals can be a high-maintenance joint by suggesting that consideration be given to modular seals that have been verified by long-term testing and designed to facilitate repair and replacement of components. Leakage, gland tears and broken welds in modular joints are common. Other States with more experience than Montana with modular joints also indicate problems on grades and in snow areas. Also, modular joints can only accommodate expansion and contraction, not rotation nor settlement.

The LRFD Specifications include no design provisions for modular joints, because they are more like mechanical assemblies than individual components required to meet specific material or allowable stress requirements.

The following will apply to the modular-type expansion joint:

1. <u>Expansion Movement</u>. The modular joint may only be considered where the anticipated expansion movement exceeds the length that can be accommodated by the strip seal expansion joints. For expansion movements greater than 125 mm, modular expansion joints may be advantageous. Its proposed use must be approved by the Bridge Design Engineer.

2. <u>Splices</u>. Where practical, modular joints should be full length with no field splices across the roadway width. If a field splice is required for traffic continuity, the support beams should be spaced at a maximum of 600 mm from the splice location. The splice will be designed according to the manufacturer's recommendations.

3. <u>Elastomeric Seal</u>. The elastomeric seal will be one piece across the roadway width, regardless of traffic continuity considerations and the presence of a field splice.

15.3.7.7 Sliding Plates

Because of maintenance and operational problems, MDT does not prefer nor widely use sliding plates. For example, if the bridge requires jacking or grade adjustments, this has caused exceptional problems where sliding plates have been used.

15.3.7.8 Example Problem

The end of Section 15.3 presents an example problem for the design of a strip seal expansion joint.

15.3.8 <u>Deck Drainage</u>

Reference: LRFD Article 2.6.6

15.3.8.1 Hydraulic Analysis

In most cases, deck drains may be located intuitively (e.g., they are not needed on short bridges or where the bridge is on a crest vertical curve). Only in rare cases will it be necessary for the Hydraulics Section to perform a hydraulic analysis for the bridge deck drainage. This may be necessary where a barrier rail is used, the bridge length exceeds 50 m to 75 m, gradients are flat and/or the roadway shoulders are narrow.

15.3.8.1 General Practices

To provide proper bridge deck drainage, the minimum longitudinal gradient is 0.2% for bridges with a barrier rail. For bridges with an open rail, the minimum is 0.0%, if there is adequate crown or superelevation to develop transverse drainage.

Adequate drainage systems shall be provided for all bridge structures. The transverse drainage of the bridge deck should be handled by providing a suitable roadway cross slope. See Section 13.5. Longitudinal drainage in a gutter section should be intercepted and not permitted to run onto the traveled way portion of the bridge. Short bridges may be constructed without drainage inlets, and the water from the bridge roadway may be transported downslope to roadway embankment protectors near the end of the bridge structure. Longitudinal drainage on long bridges shall be handled by using drainage inlets of sufficient size and number to drain the gutters adequately. For drain details, refer to the **MDT Bridge Standard Drawings**.

15.3.8.2 Downspouts

Downspouts, where required, should be of a rigid, corrosion-resistant material not less than 100 mm in diameter. Deck drainage and downspouts should be designed to prevent the discharge of drainage water against any portion of the structure and to prevent erosion at the outlet of the downspout. Also locate downspouts to avoid discharge onto traffic below or onto railroad tracks or ballast. Overhanging portions of the concrete deck shall be provided with a drip groove. For downspout details, refer to the **MDT Standard Bridge Details and Notes**.

Example Problem —Strip Seal Expansion Joint

Given: Prestressed concrete girders supporting reinforced concrete slab.

Estimated movement due to uniform temperature. Use coefficients of thermal expansion as given in LRFD Articles 5.4.2.2 and 6.4.1:

L = 70 M
$\Delta t = 80°C$
$(TU) = (70\ m)\ (80°C)\ (10.8 \times 10^{-6}/C°)\ (1000\ mm/m)$

(TU) = 60 mm (30 mm contraction, 30 mm expansion)

Note: TU is a function of the span and bearing configuration. It is equal to the length from the fixed point to the strip seal times the total change in temperature times the coefficient of thermal expansion.

Neglect creep (CR) due to elastic shortening and shrinkage (SH).

Problem: Determine expansion joint movement requirements.

Solution: Total factored movement = 1.20 (TU) (LRFD Table 3.4.1-1)

Total factored movement = 1.20 (60 mm) = 72 mm

A strip seal is acceptable because the total factored movement is within the range for strip seals (Section 15.3.7.4).

Movements from mean temperature:

Factored contraction	=	1.20 (30 mm)
	=	36 mm
Factored expansion	=	1.20 (30 mm)
	=	36 mm

Joint openings @	40°C	=	50 mm (minimum gap)
	20°C	=	68 mm (maximum installation temperature)
	0°C	=	86 mm
	-20°C	=	104 mm
	-40°C	=	50 mm (minimum gap) + 72 mm (total factored movement) = 122 mm

ΔT for 3 mm of factored movement = 3.33°C

15.4 MISCELLANEOUS STRUCTURAL ITEMS

15.4.1 Structural Design of Overhangs

Reference: LRFD Article 9.7.1.5

15.4.1.1 General

A 10-mm, double chamfer drip groove shall be placed 50 mm in from the edge of the slab. Once the controlling depth of the edge of the slab is determined, that depth should be used over the full length of the superstructure along that side of the bridge. The edge of the slab along the opposite side of a structure that is symmetrical about its centerline should be the same depth. The dimensions for the edge of slab depth are typically shown on the standard slab drawings for slabs supported by prestressed concrete beams. For the edge-of-slab depth on slabs supported by steel girders, refer to Figure 15.4A.

15.4.1.2 Width

Deck overhang width is defined as the distance from the centerline of the exterior beam to the edge of the deck. For bridges supported by prestressed concrete beams, the overhang dimensions are standardized, and that dimension is indicated on the standard slab drawings.

For bridge slabs supported on steel girders, the overhang width restrictions are the more restrictive of the following criteria:

1. not more than 0.30 to 0.35 times the beam spacing to balance moments in interior and exterior beams,
2. not more than the depth of the beam, or
3. not more than 1200 mm.

15.4.1.3 Curved Bridges

For curved bridge decks on bridges with straight girders, the limits on maximum overhang widths in Section 15.4.1.2 should be interpreted as the average within a span.

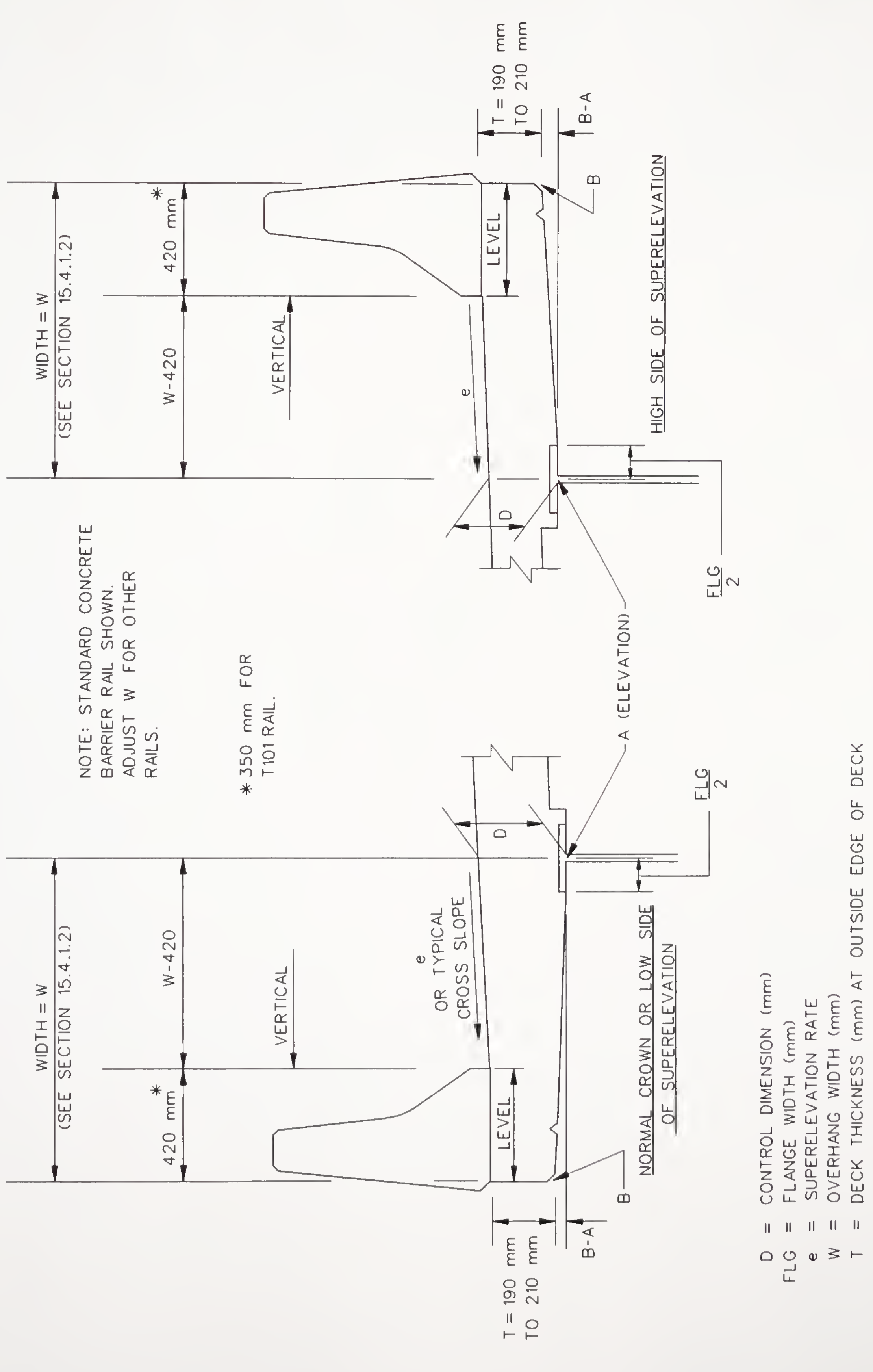
On curved deck layouts, the distance from the centerline of the girder to the edge of the slab along both sides of the slab should be shown in a Slab Offset Diagram. These offsets should be shown at tenth points measured along the centerline of the exterior girder. The offset at all break points in roadway geometrics such as beginnings of flares or turning radii along the girder line should also be shown.

15.4.1.4 Construction Considerations for Steel Girder Bridges

Because of the geometry of construction brackets used to support the overhangs, it is preferable that the bottom of the slab be made flush with the underside of the top flange on steel structures and be sloped upward 10 mm towards the edge of the slab or, at the very least, be made level. To achieve this, it may be necessary to increase Control Dimension "D," as discussed in Section 15.3.2 (Figure 15.3A), by increasing the haunch depth over the beam. If a greater Dimension "D" is established at the outside girder to control the slope of the bottom of the overhang, that dimension should be maintained at all girders throughout the structure.

15.4.1.5 Deck Depth at Outside Edge

The depth of the outside edge of the deck for steel bridges will be different than the deck thickness. See Figure 15.4A. This is true for both superelevated bridges and bridges with normal crowns. The edge of slab depth selected at each side should be maintained over the full length of the superstructure along that side.



STEEL GIRDER OVERHANG TREATMENTS

Figure 15.4A

With beam flange width known from previous beam runs and "D" established based on Section 15.3.2.2, use the following procedure with Figure 15.4A to determine the deck thickness at the outside edge of the bridge:

1. Assume a value for T.
2. Find or assume an elevation for Point A (top of exterior girder web).
3. Calculate:

   Elevation B = Elevation A + D +/– ((W - 420)e) – T

   Notes: Use 350 instead of 420 for T101 rail. Express "e" as a decimal. For bridges with normal crown sections, e = typical cross slope, usually 0.02.

4. Perform check. Elevation B – Elevation A must be between 0 and 10 (Section 15.4.1.4). If not, adjust T or D as needed to meet requirements.

15.4.1.6 Concrete Barrier

Reference: LRFD Article 3.6.1.3.4

The LRFD Specifications allows the structural contribution of any structurally continuous barrier to be used to resist transient loads at the service and fatigue-and-fracture limit states. This is typically not done in Montana but may be considered in rehabilitation if the contribution of the barrier is significant.

15.4.1.7 Collision Loads

Reference: LRFD Article A13.3.1

The design approach, as reflected by the LRFD Specifications, is that the collision loads are not specified and that the overhangs are designed for the force effects generated and transmitted by the barrier resisting the vehicular impact in a fully inelastic state. In other words, an over-design of the barrier would automatically result in an unnecessary over-design of the overhang. This over-design would increase the amount of top steel in the deck overhang. MDT experience is that slabs designed in accordance with Figure 15.2A have a long history of satisfactory performance and need not be investigated further for collision loads.

There are basically three ways by which the force effects transmitted to the overhang can be controlled:

1. reducing the barrier strength to the required minimum,
2. improving the longitudinal distribution of the collision force by barrier design, and/or
3. mitigating the transmitted force effect at the barrier-deck interface.

Such control, in conjunction with concrete barriers, can be exercised by the judicious proportioning of reinforcing steel in the barrier. Equations A13.3.1-1 through A13.3.1-4 in the LRFD Specifications indicate that the critical length of the failure pattern (L_C), which is part of the total distribution length at the barrier-to-overhang interface, increases as the longitudinal steel increases and as the transverse (vertical) steel decreases, and that the resistance of the barrier increases with "L_C".

There is a normal concentration of force effects in the end zone of the barrier, and the deck may need strengthening therein. An extension of the end beam (hidden or otherwise) to the barrier may be necessary to strengthen the overhang.

15.4.2 Design of Transverse Edge Beams

Reference: LRFD Article 9.7.1.4

For prestressed, precast girders, a transverse edge beam is required.

15.4.3 Design of Barriers

Reference: LRFD Article 13.7.3.1

Section 15.5 discusses the types of bridge rails used by the Department. Section 15.4.3 discusses the structural design of concrete and steel barriers at the edges of bridges.

15.4.3.1 Concrete

Concrete barrier railings will be built monolithically and continuous with no contraction joints at either mid-span or over the interior supports. To help control cracking, full-depth double chamfer strips are installed at 3000-mm intervals. Full-depth open joints will be provided only between the end of the structure and the concrete bridge approach and at expansion joints on structures.

Stirrups connecting any continuously placed (whether or not structurally continuous) concrete barrier, curb, parapet, sidewalk or median to the concrete decks should be determined assuming full composite action at the interface, according to the provisions of Article 5.8.4 of the LRFD Specifications.

15.4.3.2 Steel

For steel rails, the control in force effect transmission is attained by the appropriate proportioning of the post anchor bolts. In selecting the bolt material, ductility should be viewed as more important than strength. The design may facilitate the replacement of damaged bolts. Punching shear failure around the base of the post or below a concrete barrier, a brittle fracture mechanism that is difficult to rehabilitate, should be prevented. Slabs are constructed with additional reinforcement at post locations.

15.5 BRIDGE DECK APPURTENANCES

15.5.1 Bridge Rails

Reference: LRFD Article 13.7

15.5.1.1 Test Levels

Reference: LRFD Article 13.7.2

Article 13.7.2 of the LRFD Specifications identifies six test levels for bridge rails, which have been adopted from NCHRP 350 **Recommended Procedures for the Safety Performance Evaluation of Highway Features**. Test Levels One, Two, Five A, Five and Six have no application in Montana. The following identifies the general test level application for TL-3 and TL-4:

1. TL-3 (Test Level Three). Generally acceptable for a wide range of high-speed arterial highways with very low mixtures of heavy vehicles and with favorable site conditions. Performance crash testing is at 100 km/h with an 820-kg passenger car and a 2000-kg pickup truck.

2. TL-4 (Test Level Four). Generally acceptable for the majority of applications on high-speed highways, freeways, expressways, and Interstate highways with a mixture of trucks and heavy vehicles. Performance crash testing is at 100 km/h with an 820-kg passenger car and a 2000-kg pickup truck plus an 8000-kg single-unit truck at 80 km/h.

Note that, on the NHS, TL-3 is the minimum type bridge rail.

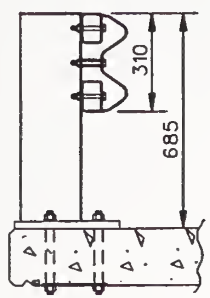
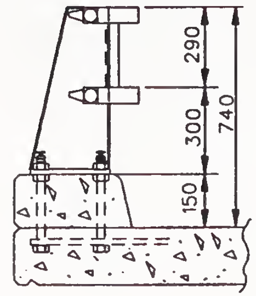
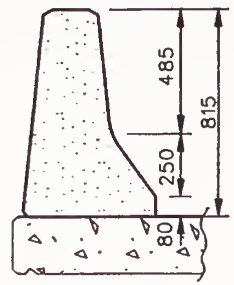
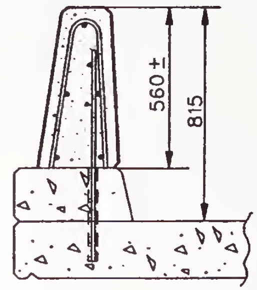
15.5.1.2 Bridge Rail Types/Usage

Figure 15.5A presents the bridge rail types approved for use by MDT. The selection of a bridge rail type will be made on a case-by-case basis considering the following factors:

1. District preference,
2. snow removal,
3. highway functional classification,
4. traffic volumes,
5. truck volumes,
6. design speed,
7. geometrics,
8. urban/rural location,
9. aesthetics,
10. in-service performance,
11. life-cycle costs,
12. consequences of rail penetration, and
13. adaptability of the guardrail-to-bridge-rail transition from the approaching roadway.

MDT has not adopted rigid criteria for the selection of bridge rail types. The following provides general guidance for those rails presented in Figure 15.5A:

1. Concrete Barrier Rail. The concrete barrier rail, which has the same face configuration as the concrete median barrier, is 815 mm in height, and its test level is TL-4. The rail's advantages include its superior performance when impacted by large vehicles, its relatively low maintenance costs and its better compatibility with the bridge deck system (i.e., the concrete rail can be constructed integrally with the bridge deck). The concrete barrier rail's disadvantages include its higher initial cost, higher dead weight and its hindrance to snow removal operations. The concrete rail is the only bridge rail allowed on current Interstate construction. It is often used on other major arterials in Montana (e.g., where the ADT >

				
System	T 101	Wyoming Curb Mounted Two-Tubed	Concrete Barrier Rail	Standard Retrofit Straight Wall
Test Level	TL-3	TL-3	TL-4	TL-4
Basic Material	Metal	Metal	Concrete	Concrete

Note: All dimensions in mm.

*Note: See the **MDT Bridge Standard Drawings** for details of the T101 and concrete barrier rails. Contact the Bridge Bureau for information on the Wyoming Curb Mounted rail and the standard retrofit straight wall.*

BRIDGE RAIL TYPES

Figure 15.5A

3000) and occasionally on urban facilities. See the **MDT Bridge Standard Drawings** for details on the concrete barrier rail design.

2. T101. The T101 bridge rail is 685 mm in height, and its test level is TL-3. The design originated in Texas and was developed and tested as the Texas 101 Rail. When compared to the concrete barrier rail, the T101's advantages include better snow removal characteristics, lower initial cost, lower dead weight and providing a more open view of the surrounding countryside. The comparative disadvantages include a lesser ability to contain heavier vehicles, higher maintenance costs and a poorer structural connection to the bridge deck system. The T101 bridge rail is often used on lower level State highways and on bridges off the State highway system. See the **MDT Bridge Standard Drawings** for details on the T101 design.

3. Wyoming Curb-Mounted Two-Tube. The Wyoming bridge rail is 740 mm in height, and its test level is TL-3. It is used in special circumstances only (e.g., where the District believes that snow removal is a special problem).

4. Brush Curbs. Where requested by the local community, a 150-mm high concrete brush curb may be used on very low-volume, low-speed bridges where the use of a traditional bridge rail would require disassembling oversized, agricultural equipment to allow its passage across the bridge.

15.5.1.3 Guardrail-To-Bridge-Rail Transitions

The Road Design Section is responsible for designing the guardrail-to-bridge-rail transition for the approaching roadway. However, site conditions may present problems for the necessary transition. Therefore, the bridge designer should ensure compatibility between the bridge rail transition and the site when selecting the bridge rail type.

15.5.1.4 Bridge Rail/Sidewalk

Reference: LRFD Articles 13.4 and 13.7.1.1

Section 13.5.4 discusses warrants for a sidewalk on a bridge. When a sidewalk is present, the location of the bridge rail requires additional consideration. Consider pedestrian safety, bridge rail performance, design speed, drainage requirements and sight distance of approaches adjacent to the ends of the bridge.

The following will apply to the location of a bridge rail in combination with a sidewalk:

1. $V \leq 70$ km/h. A sidewalk may be separated from the adjacent roadway by a barrier curb. Barrier curbs are typically 150 mm to 200 mm high with steep faces. A raised sidewalk incorporating a barrier curb is typical in urban areas with curb and gutter sections approaching the bridge. The use of a barrier curb requires a combination bridge rail and pedestrian/bicycle rail at the outside edge of the sidewalk.

2. $V \geq 80$ km/h. For high speeds, place a traffic barrier between pedestrians and traffic; i.e., between the roadway and the sidewalk. A pedestrian/bicycle rail is then used at the outside edge of the sidewalk.

Note that the total height of the combination bridge rail and pedestrian/bicycle rail adjacent to sidewalks must meet or exceed the minimum height requirements of a pedestrian rail (1060 mm) according to Article 13.8.1 of the LRFD Specifications) or bicycle rail (1100 mm according to the 1999 **AASHTO Guide for the Development of Bicycle Facilities**). The MDT has decided to standardize the 1100-mm height for both pedestrian and bicycle rails. See Figure 15.5B for the MDT typical design for extending the height of the concrete barrier rail to provide the required height for a pedestrian/bicycle rail. Bridge rails facing sidewalks should have a vertical or nearly vertical face to minimize conflicts with bicycle pedals.

15.5.2 Pedestrian Rails

Reference: LRFD Article 13.8

If a sidewalk is placed on a bridge where the design speed is greater than or equal to 80 km/h, a bridge rail shall be used to separate the vehicular traffic from pedestrians and then use a pedestrian rail on the outside edge of the sidewalk. See Section 15.5.1.4. On facilities with sidewalks on bridges and where $V \leq 70$ km/h, this arrangement will be considered on a case-by-case basis. The following factors will be evaluated:

1. design speed;
2. pedestrian volumes;
3. vehicular traffic volumes;
4. accident history;
5. geometric impacts (e.g., sight distance);
6. practicality of providing proper end treatments;
7. construction costs; and
8. local preference.

Regardless of the location of the bridge rail on bridges with sidewalks, the bridge rail shall include a pedestrian rail to a minimum height of 1100 mm. See Figure 15.5B.

Figure 15.5C presents the MDT typical design for a pedestrian rail.

15.5.3 Bicycle Rails

Reference: LRFD Article 13.9

The rail in Figure 15.5C satisfies the requirements for a bicycle rail. The following will apply:

1. Bicycle Paths. These are defined as a bikeway physically separated from motorized vehicular traffic by an open space or barrier and either within the highway right-of-way or within an independent right-of-way. Bridges which are part of a bicycle path will require a bicycle rail.

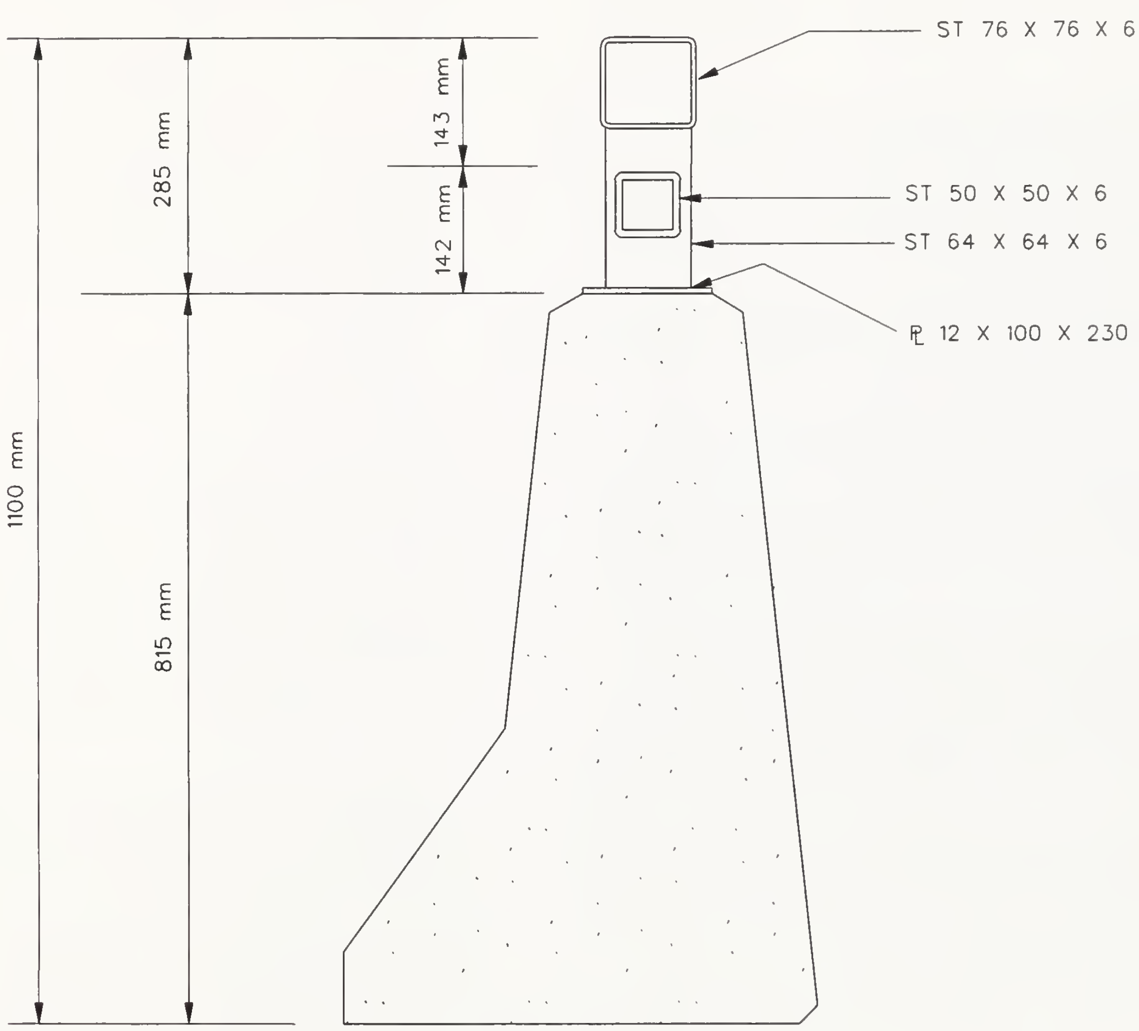
2. Other Facilities. On facilities where bicycles use the roadway with considerable frequency, it may be warranted to provide a bicycle rail across the bridge. This may either be a separate bicycle rail on the outside of the bridge where the bridge rail separates the vehicular and bicycle traffic, or a height extension to a minimum of 1100 mm on top of the bridge rail where the bridge rail is on the outside of the bridge. The need for a bicycle rail will be considered on a case-by-case basis. The following factors will be evaluated:

   a. design speed;
   b. bicycle volumes;
   c. vehicular traffic volumes;
   d. accident history;
   e. geometric impacts (e.g., sight distance);
   f. practicality of providing proper end treatments;
   g. construction costs; and
   h. local preference.

15.5.4 Fences

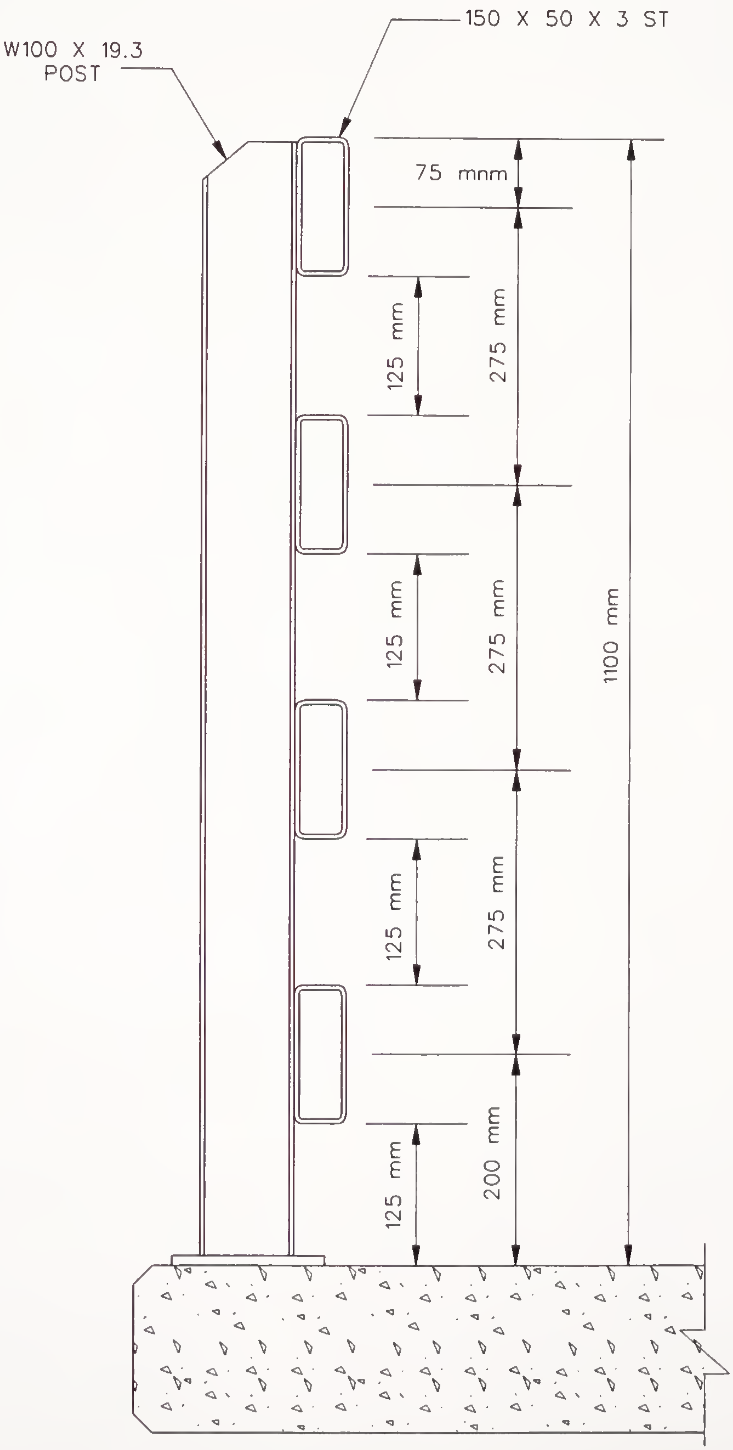
Protective fencing across bridges is warranted as follows:

1. on all overpasses in urban areas with sidewalks;
2. on other overpasses frequently used by children (e.g., near schools or playgrounds).



PEDESTRIAN/BICYCLE RAIL ON CONCRETE BARRIER RAIL

Figure 15.5B



PEDESTRIAN/BICYCLE RAIL

Figure 15.5C

See Chapter Twenty-one for fencing warrants across bridges over railroads.

Due to project differences, the connection to the barrier or rail to the fence is not standardized. The basic fence configuration is standard.

Figure 15.5D presents an example of a MDT design for fencing across bridges that illustrates the fence height and overhang.

15.5.5 Utility Attachments

The Bridge Bureau, through the Road Design Section, will coordinate with the Utilities Section within the Right-of-Way Bureau and the District for any utility attachments proposed on the bridge.

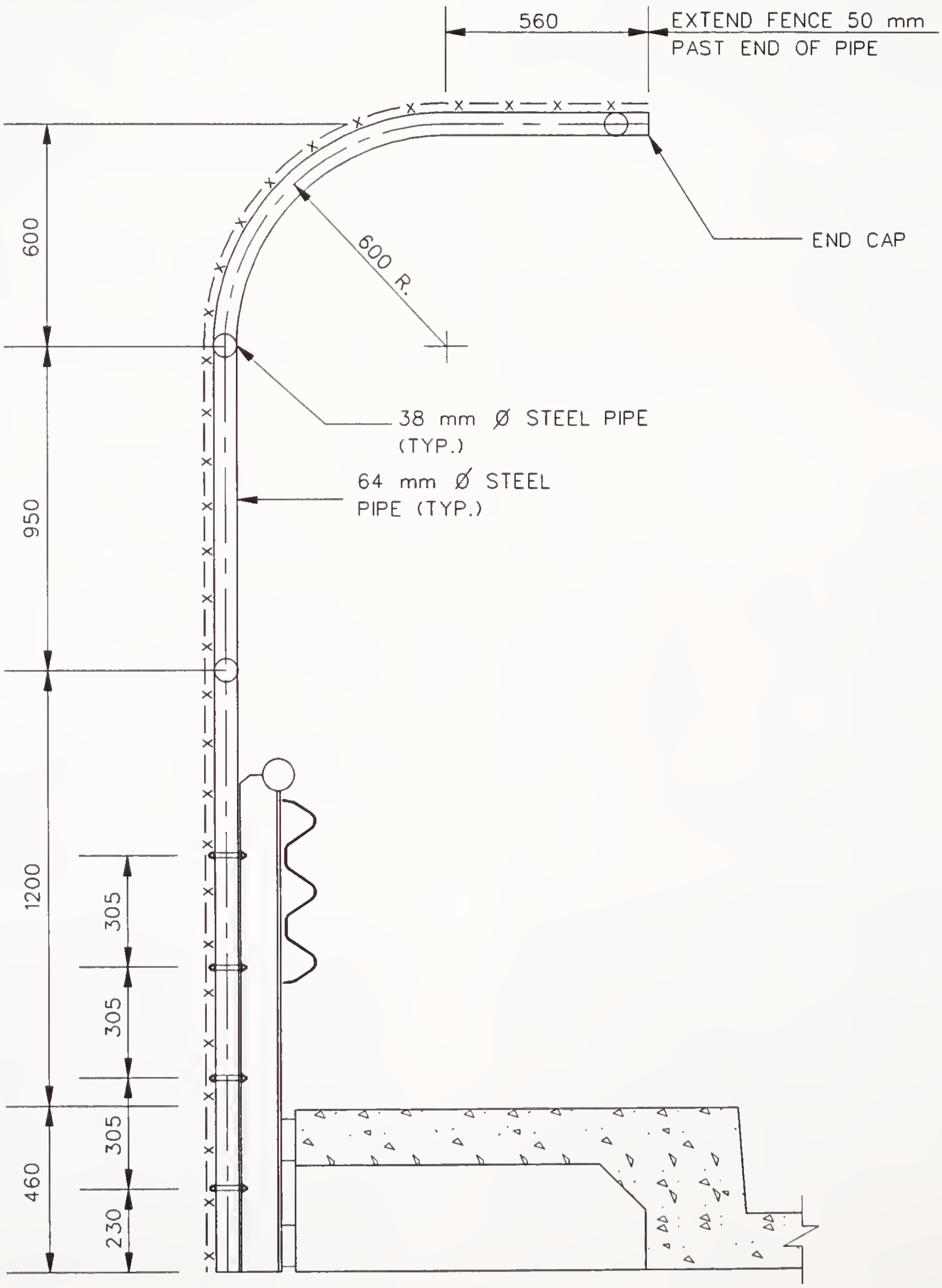
15.5.5.1 General

Utility companies frequently wish to attach utility lines or pipes to bridges, and there is a defined set of rules for consideration of their requests published in the **Administrative Rules of Montana (ARM)**. Among other issues, these rules clarify the MDT position on the attachment of utilities to a bridge. Following is a synopsis as the rules relate to bridge attachments.

The Bridge Bureau's concern is that the function of the bridge as a transportation corridor not be compromised, the safety of the individuals using the bridge not be compromised, and the Department's maintenance of the bridge not be unduly complicated. The Bridge Bureau also recognizes that existing transportation corridors offer logical routes for utilities and that, if the Department allows utility attachments to bridges, MDT can reduce costs to the Utilities and ultimately to the general public.

The following briefly explains the significant issues:

1. Utility attachments must be made in accordance with the **Administrative Rules**. The Utility cannot unilaterally hook up to a bridge because it is convenient without notifying the Department.
2. Utility attachments must be inspected and maintained. This is the Utility's responsibility, and MDT reserves the right to examine the inspection and maintenance records.
3. To ensure a safe and structurally adequate installation, MDT requires an engineered attachment plan from the Utility.
4. If the bridge cannot safely accommodate the traffic loads and the utility, the Utility will not be permitted on the bridge. Also, no attachment will be permitted that impairs MDT inspection and maintenance programs.
5. A utility attachment that reduces the vertical clearance or freeboard will not be permitted.
6. To preserve aesthetics, if a bridge is in a visible area, MDT will require that the attachment be underneath the structure, tucked in among the beams, rather than hooked to the outside.
7. To ensure a safe installation for the utility, MDT requires all attachments on the downstream side of the bridge because, during floods, trees and other drift will occasionally strike the beams.
8. MDT does not allow a utility to pass through an abutment or wingwall without specific approval; they must exit from underneath the roadway as soon as possible. Road maintenance and utility longevity are not really compatible and vice-versa.
9. The Utility will not be allowed to bolt through the deck or girders. Welding of attachments to steel members or drilling steel members will not be allowed.
10. Because MDT frequently has maintenance work on bridge rails, the Department will not allow attachments to bridge rails or the bolts used to fasten bridge rails to bridges.



FENCING ON BRIDGES

Figure 15.5D

11. Trenching operations that are so close to the bridge footings so that there may be undercutting or sloughing will not be allowed.

12. The Bridge Bureau is not the final approval authority for attachments to historic bridges; these must be cleared with other agencies as well.

13. The Utility is responsible for any damage resulting from the presence of the utility on the bridge.

14. Because certain areas in Montana are recognized earthquake areas, the Department must be critical of allowing product lines on bridges that have not been built or retrofitted to seismic design codes.

15.5.5.2 New Bridges

In addition to the above factors, the following applies to proposed utility attachments to new bridges:

1. If MDT must make a bridge stronger to support a utility, the Utility must pay for the additional design and construction costs.

2. Installation of the utility should not interfere with the MDT contractor constructing the bridge.

3. Utility facilities may pass through free-standing abutments, but not one that moves with temperature changes.

15.5.5.3 Pipelines

The following specifically applies to proposed pipelines on bridges:

1. For a pipeline installation to be approved, it must either be cased or extra strong. The design factors listed in the **ARM** are consistent with safety factors commonly used in bridge design. If the Utility proposes to meet the required design factors by using higher strength pipe, MDT will require certificates on the high-strength pipe.

2. The attachment shall be designed to prevent discharge of the pipe product into the stream or river in case of pipe failure.

3. Using bridge members to resist forces caused by moving fluids will not be permitted.

15.5.5.4 Procedures

The following briefly describes the procedures for proposed utility attachments to bridges:

1. The Utility company notifies the District Office of its desire to attach a utility to the structure and provides a proposed design for the attachment.

2. The Utility wishing to make the attachment prepares Form RW20S and submits the package to the District. After review, the District forwards the proposal to the Utility Section for transmittal to the Bridge Bureau.

3. The Bridge Bureau reviews the proposal from a structural adequacy perspective and, if in agreement, signs Form RW20S.

4. The Environmental Services Office reviews the proposal from an environmental perspective and, if in agreement, signs Form RW20S.

5. The approved utility attachment design is transmitted back to the Utility company through the District Office.

15.5.6 <u>Sign Attachments</u>

If the Signing Unit within the Traffic Engineering Section proposes to attach a sign to a bridge, the Unit must coordinate with the Bridge Bureau. The Bridge Bureau will assess the structural impact on the bridge and, if the

sign attachment is approved, the Bureau will design the attachment details.

15.5.7 Lighting/Traffic Signals

The Electrical Unit within the Traffic Engineering Section determines the warrants for highway lighting and traffic signals, and the Unit performs the design work to determine, for example, the spacing of the luminaries and the provision of electricity. In most cases, lighting will be included on bridges that are located in urban areas if requested by local officials; traffic signal warrants are determined on a case-by-case basis. Where included, the Bridge Bureau will design the structural support details for the luminaire and/or traffic signal attachments to the bridge.

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Chapter Sixteen
REINFORCED CONCRETE

Section 5 of the LRFD Bridge Design Specifications provides unified design requirements for concrete in all structural elements — reinforced, prestressed and combinations thereof. Chapter Sixteen presents MDT supplementary information specifically on the general properties of concrete and mild steel reinforcement and the design of reinforced concrete flat slabs. Chapter Seventeen discusses prestressed concrete structures.

16.1 GENERAL

16.1.1 Materials

Reference: LRFD Article 5.4.2

Class SD concrete is specified in two different compressive strengths. For the Missoula and Butte Districts, use 31 MPa. For the Billings, Great Falls and Glendive Districts, use 28 MPa unless there is documentation for the file that concrete strengths were discussed at the Design Parameters Meeting with concurrence from District and Construction staff that 31 MPa concrete is available at the project site.

Figure 16.1A presents MDT's design material properties of concrete for structural applications.

16.1.2 Strut-and-Tie Model

Reference: LRFD Article 5.6.3

Cracked reinforced concrete, reinforced with mild steel reinforcement, prestressing tendons or a combination thereof, ultimately resists load through truss-like load paths. Because reinforced concrete cracks, the compressive-stress trajectories, or struts, within the concrete tend toward straight lines. These compressive-stress trajectories plus the provided steel tensile reinforcement, or ties, form trusses. Because the concrete must be cracked, the strut-and-tie model is not applicable to the service limit states, only the strength and extreme-event limit states. A thorough presentation of the model can be found in Schlaich, J. "Towards a Consistent Design of Structural Concrete," PCI Journal, Vol. 32, No. 3, 1987.

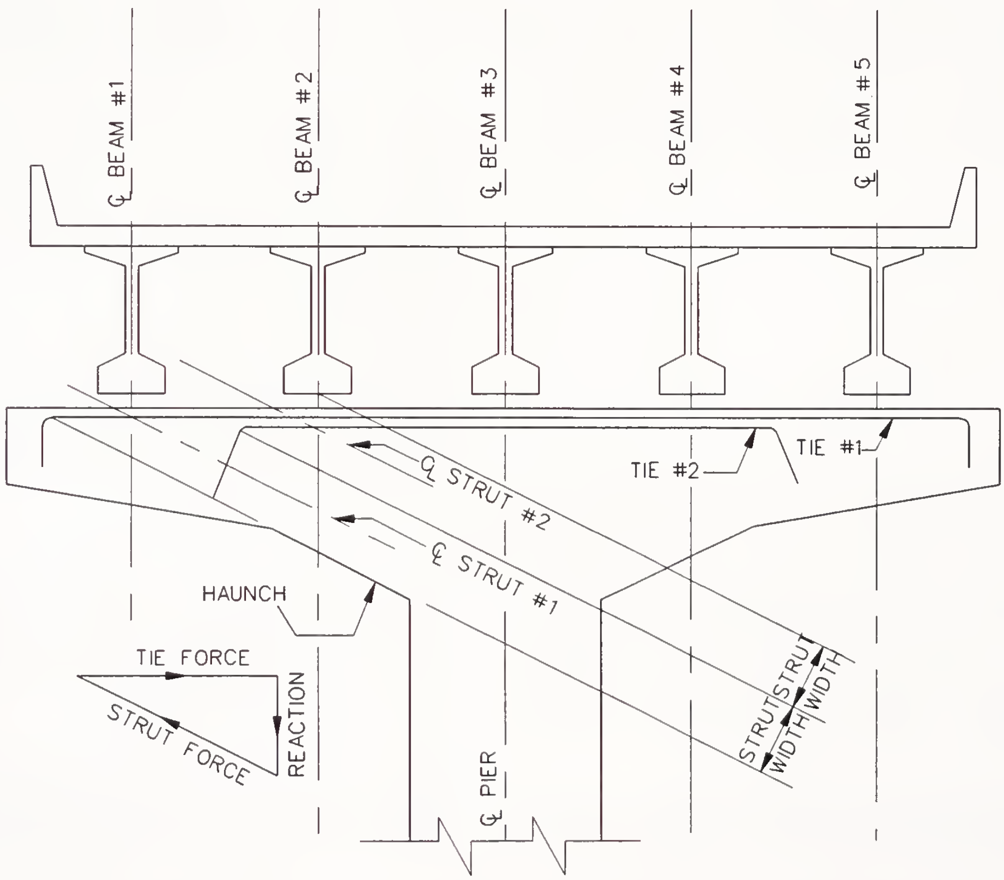
The application of the strut-and-tie model must be approved by the Bridge Area Engineer. Although the model is not typically used for actual proportioning in Montana, it can provide a fast and simple hand check.

Application of the strut-and-tie model for a hammerhead pier is demonstrated in Figure 16.1B. There are five beams supported by the

Concrete	28-day Compressive Strength (f'_c) (MPa)	Modulus of Elasticity (E_c) (MPa)	Modulus of Rupture (f_r) (MPa)
Class DD	21	22 000	2.89
Class Drilled Shaft	21	22 000	2.89
Class SD	28	25 400	3.33
Class SD	31	26 700	3.51

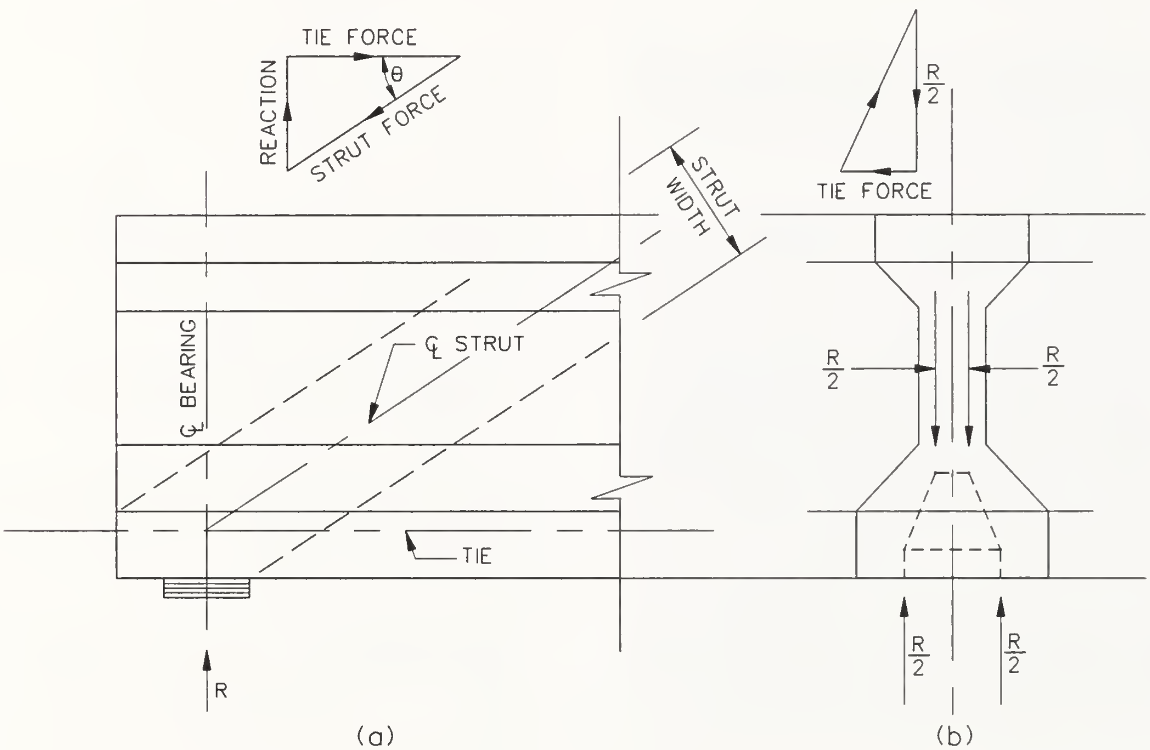
MATERIAL PROPERTIES OF CONCRETE

Figure 16.1A



STRUT-AND-TIE MODEL FOR HAMMERHEAD

Figure 16.1B



STRUT-AND-TIE MODEL FOR BEAM ENDS

Figure 16.1C

pier, of which two affect the design of a cantilever. There are several acceptable truss geometries; the one selected here ensures that the struts, being parallel, are independent from each other. The scheme is indicative of the significance of a well-proportioned cantilever. This design will yield approximately the same amount of steel in both ties. The steel in both ties is extended to the boundaries of their respective struts, then hooked down. The 90° hook of Tie #1 is further secured to the concrete by secondary steel, and the hook of Tie #2 is positioned in, and normal to, Strut #1.

This example was selected because of the potential for excessive cracking of pier heads designed as beams. Normal beam design can be unconservative for this application.

The strut-and-tie model can also be used for the approximate analysis of beam ends. Figure 16.1C(a) shows a convenient way of checking the adequacy of reinforcement in the end-zone and the magnitude of compressive stresses in the web. In lieu of refined calculations, the angle θ may be assumed as 30°. The model illustrates the futility of placing too much vertical (shear) steel in the end zone which is, except for the tie and the strut areas, largely inactive.

Figure 16.1C(b) illustrates an application of the model to estimate the transverse forces in the bearing area to be resisted by the reinforcing cage.

16.1.3 <u>Flexural Resistance</u>

Reference: LRFD Article 5.7

The general flexural-resistance equation of the LRFD Specifications (LRFD Equation 5.7.3.2.2-1) for concrete sections is rewritten below for rectangular concrete sections reinforced with mild steel reinforcement only. The nominal flexural resistance of a rectangular, singly reinforced concrete section is given as:

$$M_n = A_s f_y \left[d_s - 0.5a\right]$$

16.1.4 <u>Limits for Steel Reinforcement</u>

16.1.4.1 Maximum Reinforcement

Reference: LRFD Article 5.7.3.3.1

The LRFD Specifications unifies the maximum allowable steel reinforcement for both reinforced and prestressed sections by limiting the c/d_e ratio to 0.42. This value is based upon the same principles as the traditional limitation of 0.75 ρ_b but is applicable to all concrete sections no matter how reinforced.

16.1.4.2 Minimum Reinforcement

Reference: LRFD Article 5.7.3.3.2

The minimum flexural reinforcement on both faces of a component should provide flexural strength at least equal to the lesser of:

1. 1.2 times the cracking moment of the concrete section assuming the tensile strength as $0.63\sqrt{f'_c}$, or
2. 1.33 times the factored moment required by the governing load combination.

For a rectangular section, since:

$$\sigma = Mc/I = M/S$$

then:

$$M = \sigma S = \sigma bh^2/6$$

and the cracking moment, M_{cr}, is:

$$M_{cr} = 0.63\sqrt{f'_c}\ bh^2/6$$

$$M_{cr} = 0.105\,bh^2\sqrt{f'_c}$$

The factored resistance is:

$$M_r = 0.9\ M_n$$

but:

$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

then:

$$a = \frac{A_s f}{0.85\, f'_c\, b}$$

therefore:

$$M_n = A_s f_y \left(d - \frac{A_s f_y}{2(0.85) f'_c\, b} \right)$$

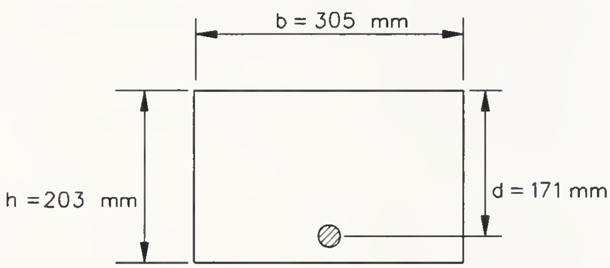
Now, the nominal flexural resistance is no longer a function of the unknown "a." Accordingly:

$$1.2 M_{cr} = 0.9 A_s f_y\, d \left[1.0 - \frac{A_s\, f_y}{1.7 b d\, f'_c} \right]$$

* * * * * * * * * *

Example 16.1.1

See Figure 16.1D for bridge section dimensions.



BRIDGE SECTION DIMENSIONS

Figure 16.1D

$b = 305$ mm, $h = 203$ mm and $f'_c = 28$ MPa:

$$M_{cr} = (0.105)(305)(203^2)\sqrt{28}$$

$$= 6983 \text{ kN - mm}$$

and $d = 171$ mm and $f_y = 420$ MPa:

$$B = \frac{(-1.7)(b d\, f'_c)}{f_y}$$

$$= \frac{(-1.7)(305)(171)(28)}{420}$$

$$= (-6937)\,\text{mm}^2$$

$$C = \frac{2.27\, M_{cr} b f'_c}{f_y^2}$$

$$= \frac{(2.533)(6983 \times 10^3)(305)(28)}{(420)^2}$$

$$= 944 \times 10^3 \text{ mm}^4$$

from which:

$$A_S = 0.5\left[-B - \sqrt{B^2 - 4C}\right] = 133 \text{ mm}^2$$

or a ratio of $\rho = 133 \div (305 \times 203) = 0.002\,15$

This process also provides the minimum steel in both directions at top and bottom of concrete flat-slab bridges.

16.1.4.3 Control of Cracking by Distribution of Reinforcement

Reference: LRFD Article 5.7.3.4

These provisions apply to all reinforced concrete flexural members except for bridge deck slabs designed in accordance with the LRFD requirements for empirical decks.

At the Service Limit State, verify that $Z/(d_c A)^{1/3} \le 0.6 f_y$ (LRFD Equation 5.7.3.4-1).

MDT has typically Severe Exposure ($Z = 23\,000$ N/mm) for bridge deck and barriers and top of hammerhead piers below expansion joints.

For all other conditions, use Moderate Exposure ($Z = 30\,000$ N/mm).

* * * * * * * * * *

16.1.5 Shear Resistance

Reference: LRFD Article 5.8

The LRFD Specifications maintains the traditional sectional approach to shear design in which the nominal shear resistance of a

reinforced concrete section is the arithmetic sum of the yield strength of the vertical steel intercepted by the critical crack and the shear resistance of the concrete. The introduction of cot θ in the equation for the steel contribution:

$$V_s = \frac{A_s f_y d}{S} \cot 0$$

signifies that the angle of inclination of diagonal compressive stresses θ could be different from the traditional 45°. In the concrete equation:

$$V_c = 0.083\,\beta\,\sqrt{f_c'}\,bd \times 10^{-3}\ \text{kN}$$

The factor "β" determines what multiple of $\sqrt{f_c'}$ may be used as the shear strength of concrete. Both "θ" and "β" are functions of the longitudinal steel strain "ε_x" which, in turn, is a function of "θ." Therefore, the design process is an iterative one. This process may be considered an improvement in accounting for the interaction between shear and flexure and trying to control cracking at strength limit state. It would appear, however, that the amount of vertical steel provided by this process is not substantially different from that given by the traditional approach. Both approaches generally yield conservative results. However, the traditional approach can be seriously unconservative for large members not containing transverse reinforcement. In typical practice, deck slabs and footings are the more common components designed without transverse reinforcement. Typically proportioned slabs and footings are not considered as large members.

The LRFD Specifications provide a simplified procedure for nonprestressed sections in LRFD Article 5.8.3.4.1 that is essentially identical to the traditional approach. This simplified procedure, wherein β and θ are assumed to be 2.0 and 45°, respectively, is the preferred procedure for reinforced concrete sections in Montana.

This simplification results in the following modifications to the equations of LRFD Article 5.8.3.3:

$$V_c = 0.166\sqrt{f_c'}\,b_v d_v$$

$$V_s = \frac{A_v f_y d_v}{S}$$

where the terms are defined in LRFD Article 5.8.3.3.

16.1.6 Fatigue Limit State

Reference: LRFD Article 5.5.3

The fatigue limit state is not normally a critical issue for reinforced concrete structures. Fatigue need not be considered for deck slabs on multiple girders or where the permanent stress f_{min} is compressive and exceeds twice the applied tensile live-load stress due to the fatigue load combination.

Assuming r/h = 0.3, the allowable stress range of LRFD Equation 5.5.3.2-1 may be rearranged for easier interpretations:

$$f_f + 0.33\,f_{min} \le 161\ \text{MPa}$$

The LRFD Specifications presents a major change in computing the applied stress range. It is the stress range due to 75% of a single truck per bridge (lane load excluded) with reduced impact and with the major axles of the truck at a constant spacing of 9 m, instead of all contributing lanes being loaded. Also, the LRFD Specifications specifies that, when the bridge is analyzed by the approximate distribution method, live-load distribution factors for one design lane loaded shall be used.

16.2 STEEL REINFORCEMENT

16.2.1 General

Reference: LRFD Article 5.4.3.1

Steel reinforcement shall consist of either uncoated, "black" rebars or epoxy-coated rebars according to these Specifications. Generally, reinforcing bars should conform to the requirements of ASTM A615/A615M, Grade 420 with a 420 MPa yield strength. For seismic applications, rebars conforming to ASTM A706 should be specified for greater quality control of unanticipated overstrength.

16.2.2 Sizes

Reinforcing bars are referred to in the contract plans and specifications by number, and they vary in size from #13 to #57. Figure 16.2A shows the sizes and various properties of the typical bars used in Montana.

To avoid handling damage, the minimum bar size shall be #13.

16.2.3 Lengths

To facilitate handling, the maximum length of #13 reinforcing bars is 12.19 m. Larger bars can be specified in lengths up to 18.29 m.

16.2.4 Concrete Cover

Reference: LRFD Article 5.12.3

See Figure 16.2B for MDT criteria for minimum concrete cover for various applications. All clearances to reinforcing steel shall be shown on the plans.

16.2.5 Spacing of Reinforcement

Reference: LRFD Article 5.10.3

For minimum spacing of bars, see Figure 16.2C. Fit and clearance of reinforcing shall be carefully checked by calculations and large-scale drawings. Skews tend to aggravate problems of reinforcing interference. Tolerances normally allowed for cutting, bending and locating reinforcing shall be considered.

Common areas of interference are:

Bar Size Designation	Nominal Dimensions		
	Mass (kg/m)	Diameter (mm)	Area (mm^2)
#13	0.994	12.7	129
#16	1.552	15.9	199
#19	2.235	19.1	284
#22	3.042	22.2	387
#25	3.973	25.4	510
#29	5.060	28.7	645
#32	6.404	32.3	819
#36	7.907	35.8	1006
#43	11.38	43.0	1452
#57	20.24	57.3	2581

REINFORCEMENT BAR PROPERTIES

Figure 16.2A

Item	Minimum Concrete Cover for Design & Detailing (mm)
Deck Slabs (including both slabs on girders and flat-slab bridges)	
Top Steel	60
Bottom Steel	25
Footings and Pier Shafts	50
Stirrups and Ties	40
Items cast against ground	80
Drilled Shaft	75
All Other Structural Elements	50

CONCRETE COVER

Figure 16.2B

Bar Size	Preferred Minimum Spacing (mm)	
	Unspliced Bars	Spliced Bars
#13	50	65
#16	54	70
#19	58	78
#22	60	82
#25	64	90
#29	72	100
#32	80	112
#36	90	N/A
#43	108	N/A
#57	143	N/A

Note: *Minimum spacing values are based upon Articles 5.10.3.1.1 and 5.10.3.1.4 in the LRFD Specifications and the nominal diameters of metric reinforcing bars. In Montana, the maximum size of coarse aggregate used in both cast-in-place and precast concrete is 19 mm.*

MINIMUM SPACING OF BARS

Figure 16.2C

1. between slab reinforcing and reinforcing in monolithic end bents or intermediate bents;
2. vertical column bars projecting through main reinforcing in pier caps;
3. the areas near expansion devices;
4. anchor plates for steel girders; and
5. between prestressing steel and reinforcing steel stirrups, ties, etc.

Show the clear distance from the face of concrete to the first bar. When the distance between the first and last bars is such that the number of bars required results in spacings in increments of other than 5 mm, show the bars to be equally spaced.

16.2.6 Development of Reinforcement

Reference: LRFD Article 5.11.2

Reinforcement is required to be developed on both sides of a point of maximum stress at any section of a reinforced concrete member. This requirement is specified in terms of a development length, l_d.

16.2.6.1 Development Length in Tension

Development of bars in tension involves calculating the basic development length, l_{db}. The development length is modified by factors to reflect bar spacing, cover, enclosing transverse reinforcement, top bar effect, type of aggregate, epoxy coating and the ratio of the required area to the provided area of reinforcement.

The development length, l_d (including all applicable modification factors) must not be less than 300 mm.

Figures 16.2D through 16.2G show the tension development length for both uncoated and epoxy-coated Grade 420 bars for normal weight concrete with specified strengths of 21 and 28 MPa. For Class SD concrete with $f'_c = 31$ MPa, use development lengths shown for $f'_c = 28$ MPa.

16.2.6.2 Development Length in Compression

The standard procedure in Montana is to use tension development lengths for bars in either tension or in compression. This ensures that an adequate development length will be provided in a compression member that may be primarily controlled by bending. Hooks are not considered effective in developing bars in compression. When designing column bars with hooks to develop the tension, ensure that the straight length is also an adequate length to develop the bar in compression.

16.2.6.3 Standard End Hook Development Length in Tension

Standard end hooks, utilizing 90- and 180-degree end hooks, are used to develop bars in tension where space limitations restrict the use of straight bars. End hooks on compression bars are not effective for development length purposes. The values shown in Figures 16.2H through 16.2J show the tension development lengths for normal weight concrete with specified strengths of 21 and 28 MPa. For Class SD concrete with $f'_c = 31$ MPa, use development lengths shown for $f'_c = 28$ MPa.

Figure 16.2K illustrates the hooked-bar details for the development of standard hooks.

Figure 16.2D
Tension Development Lengths(l_d) for Grade 420 Uncoated Bars; f'_c = 21 MPa; Normal Weight Concrete

Bar Size	l_d		l_d Modified for Bar Spacing Per Article 5.11.2.1.3 (Spacing ≥ 150 mm)	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	450	320	360	260
#16	570	410	450	330
#19	730	530	590	420
#22	1000	710	800	570
#25	1310	940	1050	750
#29	1660	1190	1330	950
#32	2110	1510	1690	1210
#36	2590	1850	2070	1480
#43	3210	2300	2570	1840
#57	4370	3120	3490	2500

Figure 16.2E
Tension Development Lengths (l_d) for Grade 420 Uncoated Bars; f'_c = 28 MPa; Normal Weight Concrete

Bar Size	l_d		l_d Modified for Bar Spacing Per Article 5.11.2.1.3 (Spacing ≥ 150 mm)	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	450	320	360	260
#16	570	410	450	330
#19	680	490	540	390
#22	860	620	690	500
#25	1140	810	910	650
#29	1440	1030	1150	820
#32	1830	1310	1460	1050
#36	2240	1600	1790	1280
#43	2780	1990	2230	1590
#57	3780	2700	3030	2160

STRAIGHT UNCOATED DEFORMED BARS

Figure 16.2F
Tension Development Lengths (l_d) for Grade 420 Epoxy Coated Bars; f'_c = 21 MPa; Normal Weight Concrete

Bar Size	l_d		l_d Modified for Bar Spacing Per Article 5.11.2.1.3 (Spacing ≥ 150 mm)	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	540	390	430	310
#16	680	490	540	390
#19	880	630	700	500
#22	1200	860	960	690
#25	1580	1130	1260	900
#29	1990	1420	1590	1140
#32	2530	1810	2200	1450
#36	3100	2220	2480	1770
#43	3850	2750	3080	2200
#57	5240	3740	4190	3000

Figure 16.2G
Tension Development Lengths (l_d) for Grade 420 Epoxy Coated Bars; f'_c = 28 MPa; Normal Weight Concrete

Bar Size	l_d		l_d Modified for Bar Spacing Per Article 5.11.2.1.3 (Spacing ≥ 150 mm)	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	540	390	430	310
#16	680	490	540	390
#19	810	580	650	470
#22	1040	740	830	590
#25	1370	980	1090	780
#29	1720	1230	1340	990
#32	2190	1570	1750	1250
#36	2690	1920	2150	1540
#43	3340	2390	2670	1910
#57	4540	3240	3630	2600

STRAIGHT EPOXY COATED DEFORMED BARS

Note: The shaded cells indicate where the tabularized tension development lengths do not meet the compressive development length requirements of LRFD Article 5.11.2.2.1.

Figure 16.2H
Tension Development Lengths(l_{dh}) Grade 420
Uncoated Bars; f'_c = 21 MPa; Normal Weight Concrete

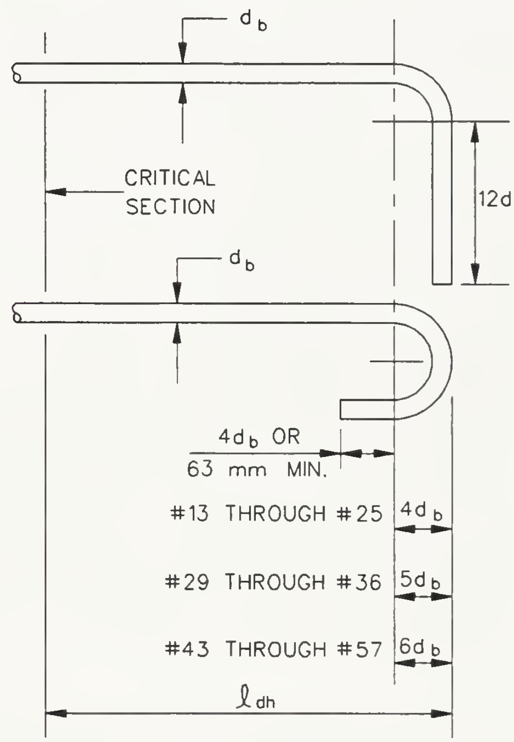
Bar Size	l_{dh} Side Cover < 60 mm or Cover on Tail <50 mm $l_{dh} = l_{db}$ (mm)	l_{dh} Side Cover ≥ 60 mm and Cover on Tail ≥ 50 mm $l_{dh} = 0.7\ l_{db}$ (mm)
#13	280	200
#16	350	250
#19	420	300
#22	490	350
#25	560	400
#29	630	450
#32	710	500
#36	790	560
#43	950	950
#57	1260	1260

Figure 16.2I
Tension Development Lengths (l_{dh}) Grade 420
Uncoated Bars; f'_c = 28 MPa; Normal Weight Concrete

Bar Size	l_{dh} Side Cover < 60 mm or Cover on Tail <50 mm $l_{dh} = l_{db}$ (mm)	l_{dh} Side Cover ≥ 60 mm and Cover on Tail ≥ 50 mm $l_{dh} = 0.7\ l_{db}$ (mm)
#13	250	180
#16	310	220
#19	370	260
#22	430	310
#25	490	350
#29	550	390
#32	620	440
#36	690	490
#43	820	820
#57	1090	1090

Figure 16.2J
Hook Lengths After Bends

Bar Size	Hook Length After 90° Bend (mm)	Hook Length After 180° Bend (mm)
#13	160	70
#16	200	70
#19	230	80
#22	270	90
#25	310	110
#29	350	120
#32	390	130
#36	430	150
#43	520	180
#57	690	230



Hooked-Bar Details for Development of Standard Hooks
Figure 16.2K

Note: *Development lengths shown to be multiplied by a factor of 1.2 for epoxy coated bars.*

16.2.7 Splices

Reference: LRFD Article 5.11.5

16.2.7.1 General

Three methods may be used to splice reinforcing bars — lap splices, mechanical splices and welded splices. Lap splicing of reinforcing bars is the most common method. Lap splices are not allowed in potential plastic hinge regions. To minimize the possibility of mislocated lap splices, the plans should clearly show the locations and lengths of all lap splices. Due to splice lengths required, lap splices are not permitted for bars larger than #36; if bars larger than #36 are necessary, mechanical bar splices shall be used.

No lap splices, for either tension or compression bars, shall be less than 310 mm.

If transverse reinforcing steel in a bridge deck will be lapped near a longitudinal construction joint, show the entire lap splice on the side of the construction joint that will be poured last.

16.2.7.2 Lap Splices — Tension

Reference: LRFD Article 5.11.5.3

Many of the same factors which affect development length affect splices. Consequently, tension lap splices are a function of the bar development length (l_d). All lap splices in tension should be detailed as Class C tension lap splices unless problems arise. The other lesser classes of LRFD Article 5.11.5.3.1 may be used only if the requirements of Class C cannot be satisfied. Designers are encouraged to splice bars at points of minimum stress.

For tension splices, the length of a lap splice between bars of different sizes shall be governed by the smaller bar.

Figures 16.2L through 16.2O show tension lap splices for both uncoated and epoxy coated Grade 420 bars for normal weight concrete with specified strengths of 21 and 28 MPa. For Class SD concrete with specified strength of 31 MPa, use the tabularized values for 28 MPa.

16.2.7.3 Lap Splices — Compression

In Montana, lap splices in compression members are sized for tension lap splices. The design of compression members, such as columns, pier walls and abutment walls, involves the combination of vertical and lateral loads. The policy of requiring a tension lap splice considers the possibility that the member may be primarily controlled by bending. The increase in cost of additional splice reinforcement material is minimal.

16.2.7.4 Mechanical Splices

A second method of splicing is by mechanical splices, which are proprietary splicing mechanisms. Mechanical splices are appropriate where interference problems preclude the use of more conventional lap splices and in phased construction. Even with mechanical splices, it is frequently necessary to stagger splices. The designer must check clearances. The requirements for mechanical splices are found in Articles 5.11.5.2.2, 5.11.5.3.2 and 5.11.5.5.2 of the LRFD Specifications. Epoxy-coated mechanical splices must be used with epoxy-coated reinforcing steel.

16.2.7.5 Welded Splices

Splicing of reinforcing bars by welding, although allowed by the LRFD Specifications, is seldom used by MDT and not encouraged principally because of quality issues with field welding. However, it is common practice to weld the tail of a column spiral reinforcing back on itself. According to the LRFD Specifications within plastic hinge zones, reinforcing steel splices are limited to welded splices or mechanical connectors. Those provisions of the Specifications may make welded splices the best

choice in certain circumstances. Welding reinforcing steel is not covered by the **AASHTO/ANSI/AWS D1.5 Bridge Welding Code**, and the current **Structural Welding Code — Reinforcing Steel** of AWS (D1.4) must be referenced. The AASHTO Code does not allow welded splices in decks.

All reinforcing steel used by MDT can be welded; however, if reinforcing steel is to be welded, A-706M reinforcing steel is preferred due to tighter controls on the carbon content. The carbon content determines preheat requirements for welding.

16.2.8 Epoxy-Coated Reinforcement

Reference: LRFD Articles 2.5.2.1.1 and 5.12.4

MDT uses epoxy-coated reinforcement at the following locations:

1. all bridge deck reinforcement;
2. all reinforcing that extend into the slabs, including cast-in-place concrete diaphragm shear steel and excluding prestressed girder shear connectors;
3. vertical back wall and back wall connection steel extending into the slab for structures located on the State highway system (not for off-system bridges);
4. cap shear and primary reinforcement of caps located under deck expansion joints. Also include beam seat and shear block reinforcement at these locations; and
5. all reinforcing in bridge approach slabs.

For other locations, use plain reinforcing steel. For example:

1. bridge deck and deck joint rehabilitations of existing bridges with plain steel;
2. all substructure reinforcement including footings, piers, columns and caps not specifically identified above as needing epoxy bars;
3. pile reinforcing at pipe-pile-to-cap connections, except for integral caps where the vertical bars extend into the limits of the slab;
4. wing wall reinforcement for typical and turnback wings; and
5. reinforcing for typical reinforced concrete retaining walls.

16.2.9 Detailing of Reinforcement

16.2.9.1 Spirals

Figure 16.2P illustrates the detailing of spiral reinforcement. In the Bill of Reinforcing both the height of the spiral and the length of the bent bar should be indicated plus the pitch spacing and spiral radius.

Figure 16.2L
Tension Lap Splice Lengths for Grade 420 Uncoated Bars; f'_c = 21 MPa; Normal Weight Concrete

Bar Size	Center-to-Center Spacing < 150 mm or < 75 mm from side face of member		Center-to-Center Spacing ≥ 150 mm and ≥ 75 mm from side face of member	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	750	530	620	430
#16	940	670	750	550
#19	1180	840	960	680
#22	1590	1130	1280	910
#25	2080	1480	1670	1190
#29	2620	1870	2120	1500
#32	3340	2380	2670	1910
#36	4140	2950	3320	2370
#43	Lap Splices Not Allowed			
#57				

Figure 16.2M
Tension Lap Splice Lengths for Grade 420 Uncoated Bars; f'_c = 28 MPa; Normal Weight Concrete

Bar Size	Center-to-Center Spacing < 150 mm or < 75 mm from side face of member		Center-to-Center Spacing ≥ 150 mm and ≥ 75 mm from side face of member	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	750	530	620	430
#16	940	670	750	550
#19	1110	790	890	630
#22	1400	990	1130	800
#25	1790	1280	1430	1020
#29	2300	1640	1840	1310
#32	2890	2060	2320	1650
#36	3560	2540	2860	2040
#43	Lap Splices Not Allowed			
#57				

CLASS C — Uncoated Bars

Figure 16.2N
Tension Lap Splice Lengths for Grade 420 Epoxy Coated Bars; f'_c = 21 MPa; Normal Weight Concrete
(Bar cover ≥ $3d_b$ and clear spacing between bars ≥ $6d_b$)

Bar Size	Center-to-Center Spacing < 150 mm or < 75 mm from side face of member		Center-to-Center Spacing ≥ 150 mm and ≥ 75 mm from side face of member	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	910	650	750	510
#16	1130	800	910	670
#19	1420	1010	1160	820
#22	1910	1360	1530	1090
#25	2500	1790	2010	1430
#29	3150	2250	2540	1810
#32	4020	2860	3220	2300
#36	4970	3540	3980	2840
#43	Lap Splices Not Allowed			
#57				

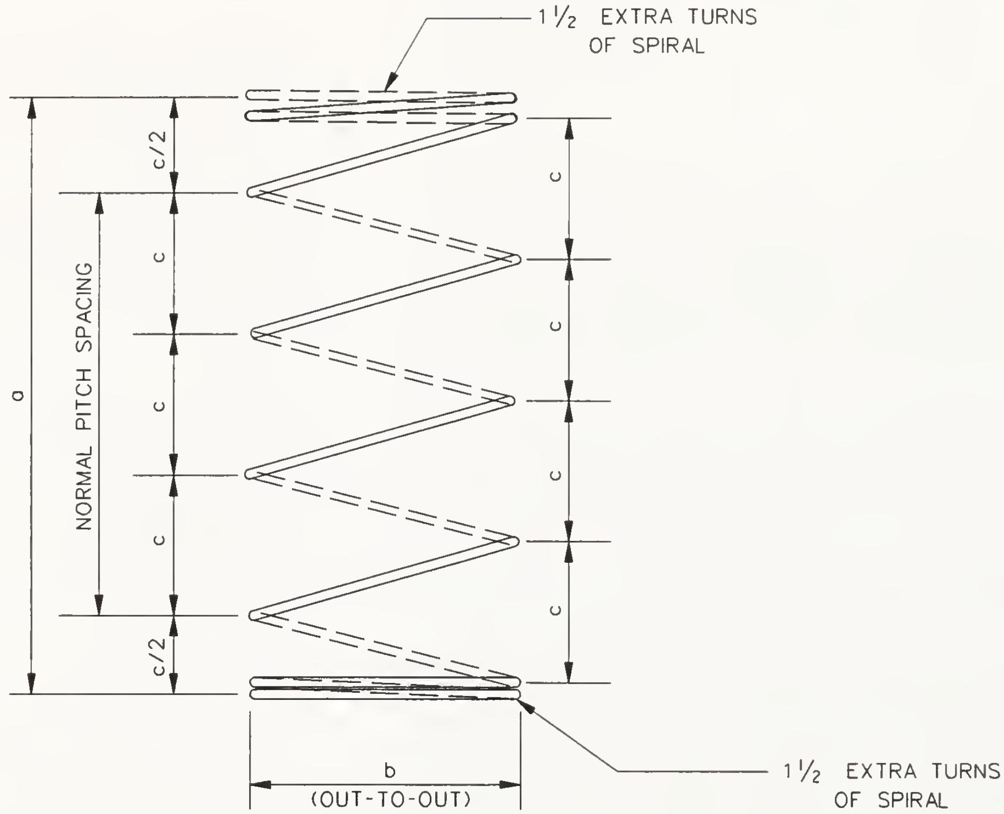
Figure 16.2O
Tension Lap Splice Lengths for Grade 420 Epoxy Coated Bars; f'_c = 28 MPa; Normal Weight Concrete
(Bar cover ≥ $3d_b$ and clear spacing between bars ≥ $6d_b$)

Bar Size	Center-to-Center Spacing < 150 mm or < 75 mm from side face of member		Center-to-Center Spacing ≥ 150 mm and ≥ 75 mm from side face of member	
	Top Bars (mm)	Others (mm)	Top Bars (mm)	Others (mm)
#13	910	650	750	510
#16	1130	800	910	670
#19	1330	960	1080	770
#22	1690	1190	1360	970
#25	2150	1530	1720	1230
#29	2760	1980	2210	1590
#32	3470	2490	2790	1990
#36	4270	3050	3440	2450
#43	Lap Splices Not Allowed			
#57				

CLASS C — Epoxy-Coated Bars

Top bars are horizontal bars so placed that more than 300 mm of fresh concrete is cast in the member below the bar.

Splice lengths shown in the Figures for both uncoated and epoxy-coated bars must be multiplied by a factor of 2.0 for bars with a cover of d_b or less, or with a clear spacing between bars of $2d_b$ or less, where d_b equals the bar diameter.



KEY:

a = SPIRAL HEIGHT
b = OUTSIDE DIAMETER
c = PITCH
d = BAR DIAMETER
L = TOTAL LENGTH OF SPIRAL REINFORCEMENT

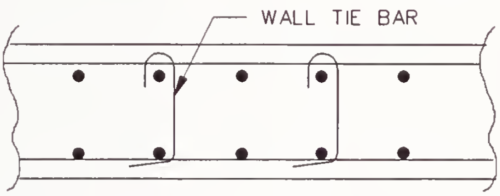
$L = [(a/c + 2(1\frac{1}{2}\text{ TURNS})]\pi b$

SPIRAL REINFORCEMENT

Figure 16.2P

16.2.9.2 Wall Tie Bars

Use wall tie bars with a 180° hook on one end and a 90° hook on the other. This deviation from the LRFD Specifications is allowed to facilitate placement of the ties. See Figure 16.2Q for an example of a wall tie bar.



WALL PLAN VIEW
Figure 16.2Q

16.2.9.3 Drilled Shaft Cages

Drilled shaft reinforcing cages have often in the past been detailed with cross ties. The purpose of these ties is to stabilize the cage during placement of the cage into the shaft. Experience has shown that cross ties frequently make concrete placement more difficult. Because the stability of the reinforcing cage is the contractor's responsibility and cross ties are only used during construction, it is not necessary to show them on the plans.

To assure good concrete flow through the reinforcing cage, the clear spacing of all bars should not be less than 5 times the coarse aggregate size. To meet this requirement, bundled bars are allowed. This spacing requirement applies universally, whether in splices, spirals or hoops or among the main reinforcement. Use lap or mechanical splices and place splices at the lower end of the cage to provide lower splice stresses due to service loads and cage handling. Stagger mechanical splices in adjacent bars a minimum of 600 mm. Be aware that using hooks at the top ends of vertical reinforcement can complicate casing extraction and the placement of cap cages. Hook orientation should be determined by the design engineer and shown on the plans.

16.3 REINFORCED CAST-IN-PLACE CONCRETE FLAT SLABS

16.3.1 General

Reference: LRFD Article 5.14.4

The superstructures typically called "flat slabs" in Montana are termed "slab superstructures" in the LRFD Specifications.

The reinforced cast-in-place concrete flat slab bridge is frequently used by MDT because of its suitability for short spans and its ease of construction. It is the simplest among all superstructure systems.

Section 16.3 presents information for the design of reinforced cast-in-place concrete flat slabs that amplify or clarify the provisions in the LRFD Bridge Design Specifications. The Section also presents design information specific to MDT practices.

16.3.1.1 Materials

Reference: LRFD Article 5.4

Use Class SD concrete for reinforced concrete flat slabs. See Figure 16.1A for concrete properties.

16.3.1.2 Cover

Reference: LRFD Article 5.12.3

Figure 16.3A presents MDT criteria for minimum concrete cover for various elements of reinforced concrete flat slabs. All clearances to reinforcing steel shall be shown on the plans.

16.3.1.3 Haunches

In general, MDT prefers straight haunches over parabolic haunches because straight haunches are comparatively easy to form yet result in relatively good stress flow.

Haunching is used to decrease maximum positive moments in continuous structures by attracting more negative moments to the haunches and to provide adequate resistance at the haunches for the increased negative moments. It is a simple, effective and economical way to enhance the resistance of thin concrete flat slabs.

The preferable ratio between the end and intermediate spans is approximately 0.75 to 0.80, but the system permits considerable freedom in selecting span ratios. The ratio between the depths at the edge of intermediate pier cap and at the point of maximum positive moment should be approximately 1.2. Except for aesthetic reasons, the length of the haunch should be approximately 0.15L, where "L" is the intermediate span length; longer haunches may

Element	Minimum Concrete Cover
Top of Slab	60 mm*
Bottom of Slab	25 mm
Ends of Slab	40 mm
Edge of Slab	75 mm

**This includes a 35-mm sacrificial wearing surface.*

MINIMUM CONCRETE COVER
(Reinforced Concrete Flat Slabs)

Figure 16.3A

be unnecessarily expensive and/or structurally counterproductive.

16.3.1.4 Minimum Reinforcement

Reference: LRFD Articles 5.7.3.3.2, 5.10.6 and 5.14.4.1

In both the longitudinal and transverse directions, at both the top and bottom of the slab, the minimum reinforcement should be determined according to the provisions of Articles 5.7.3.3.2 and 5.10.8 in the LRFD Specifications. The first is based on the cracking flexural strength of a component, and the second reflects requirements for shrinkage and temperature. In flat slabs, the two articles provide nearly identical amounts of minimum reinforcement in the majority of cases.

According to Article 5.14.4.1 of the LRFD Specifications, bottom transverse reinforcement, the above-minimum provisions notwithstanding, may be determined either by two-dimensional analysis or as a percentage of the maximum longitudinal positive moment steel in accordance with LRFD Equation 5.14.4.1-1. The span length, L, in the equation should be taken as that measured from the centerline to centerline of the supports. Especially for heavily skewed and/or curved bridges, the analytical approach is recommended.

Section 16.3.4 gives a simplified approach for shrinkage and temperature steel requirements.

16.3.2 Construction Joints

Transverse construction joints are not allowed for design purposes in reinforced concrete flat slabs. However, because of construction problems, they may become unavoidable. The **MDT Standard Specifications** provides construction requirements where transverse construction joints are unavoidable.

Longitudinal construction joints in reinforced concrete slab bridges are also undesirable. However, bridge width, phase construction, the method of placing concrete, rate of delivery of concrete, and the type of finishing machine used by the contractor may dictate whether or not a reinforced concrete slab bridge can be poured in a single pour.

If the slab structure will be built in phases, show the entire lap splice for all transverse reinforcing steel on the side of the construction joint that will be poured last.

16.3.3 Longitudinal Edge Beam Design

Reference: LRFD Articles 5.14.4.1, 9.7.1.4 and 4.6.2.1.4

Edge beams must be provided along the edges of flat slabs. The edge beams can be thickened sections and/or more heavily reinforced sections composite with the slab. The width of the edge beams may be taken to be the width of the equivalent strip used in analysis as described in Section 16.3.8.

16.3.4 Shrinkage and Temperature Reinforcement

Reference: LRFD Article 5.6.2

MDT practice is that evaluating the redistribution of force effects as a result of shrinkage, temperature change, creep and movements of supports is not necessary when designing reinforced concrete flat slabs.

Shrinkage and temperature reinforcement is #13s @ 300 mm.

16.3.5 Reinforcing Steel and Constructibility

The following practices for reinforcing steel should be met to improve the constructibility of reinforced concrete flat slabs:

1. The maximum reinforcing bar size shall be #36.

2. The minimum spacing of reinforcing bars shall be 100 mm.

16.3.6 Deck Drainage

Reference: LRFD Article 2.6.6

Section 15.3.8 discusses drainage for bridge decks in conjunction with prestressed concrete or structural steel superstructures. This information also applies to reinforced concrete flat slabs.

16.3.7 Distribution of Concrete Barrier Railing Dead Load

The edge beam carries the dead load of the barrier.

16.3.8 Distribution of Live Load

Reference: LRFD Articles 4.6.2.3 and 4.6.2.1.4

Section 14.3.2 discusses the application of vehicular live load, and Section 15.2 discusses the application of the Strip Method to bridge decks. The following specifically applies to the distribution of live load to reinforced concrete flat slabs:

1. For continuous flat slabs with variable span lengths, one equivalent strip width (E) shall be developed using the shortest span length for the value of L_1. This strip width should be used for moments throughout the entire length of the bridge.

2. The equivalent strip width (E) is the transverse width of slab over which an "axle" unit is distributed.

3. Different strip widths are specified for the flat slab itself and its edge girders in LRFD Articles 4.6.2.3 and 4.6.2.1.4, respectively.

4. In most cases, using Equation 4.6.2.3-3 from the LRFD Specifications for the reduction of moments in skewed slab-type bridges will not significantly change the reinforcing steel requirements. Therefore, for simplicity of design, the Department does not require the use of the reduction factor "r."

16.3.9 Shear Resistance

Reference: LRFD Article 5.14.4.1

Single-span and continuous-span flat slabs designed for moment in conformance with Article 4.6.2.3 of the LRFD Specifications may be considered satisfactory for shear.

16.3.10 Minimum Thickness of Slab

Reference: LRFD Article 2.5.2.6.3

For the typical MDT three-span continuous flat-slab bridges of total lengths of 18 300 mm and 23 775 mm, the minimum slab thicknesses are 360 mm and 410 mm, respectively. These specified minimum thicknesses include a 35-mm sacrificial wearing surface.

In the event of clearance problems and with the approval of the Crew Chief, the minimum slab thickness requirements in accordance with LRFD Table 2.5.2.6.3-1 may be used. In using the equations in the LRFD Table, it is assumed that:

1. S is the length of the longest span.

2. The calculated thickness includes the 35-mm sacrificial wearing surface.

3. The thickness used may be greater than the minimum if needed for design.

16.3.11 Development of Flexural Reinforcement

Reference: LRFD Article 5.11.1.2

Article 5.11.1.2 of the LRFD Specifications presents specifications for the portion of the longitudinal positive moment reinforcement that must be extended past the point required by the factored maximum moment diagram. Similarly, there is a more stringent provision addressing the location of the anchorage for the longitudinal negative moment reinforcement.

16.3.12 Skews on Reinforced Concrete Slab Bridges

Reference: LRFD Article 9.7.1.3

For up to a 25° skew angle, the transverse reinforcement is permitted to run parallel to the skew, providing for equal bar lengths. In excess of 25°, the transverse reinforcement should be placed perpendicular to the longitudinal reinforcement. This provision concerns the direction of principal tensile stresses as they develop in heavily skewed structures and is intended to prevent excessive cracking.

16.3.13 Substructures

The following describes typical MDT practice for types of substructures used in conjunction with reinforced concrete flat slabs:

1. End Supports. Where possible, use integral or semi-integral abutments. In general, their use is not restricted by highway alignment nor skew; the maximum length is 60 m for use of integral abutments without a special analysis. See Sections 13.4 and 19.1 for more information on end supports, including the use of non-integral abutments and the use of integral abutments where the bridge length exceeds 60 m.

2. Intermediate Supports. See Section 19.2 for typical MDT practices for the selection of the type of intermediate support (e.g., wall piers, pipe pile bents and multiple column bents).

16.3.13.1 Design Details for Integral Caps at Intermediate Bents of Flat Slabs

The following presents specific design details which represent typical MDT practices for the design of integral caps in conjunction with reinforced concrete flat slabs:

1. MDT's standard cap dimensions are 1000 mm in width and a depth of 1000 mm from the top of the slab to the break point in the crown.

2. All shear reinforcement in the caps is placed parallel to the longitudinal slab steel.

3. Standard pile embedment into the abutment and intermediate bent caps is 500 mm.

4. A 10-mm drip groove shall be located 50 mm in from the edge of the slab.

5. The profile of the bottom of the bent cap can be made level if the difference in top-of-slab elevations at the left and right edge of slab, along the centerline of the bent cap, is 75 mm or less. For a difference of greater than 75 mm, slope the bottom of the cap.

Figure 16.3C illustrates a typical section of an integral cap at an intermediate bent of a flat slab.

16.3.13.2 Design Details for End Bents of Flat Slabs

Flat slab end bents typically consist of a pile cap with a backwall constructed above it. The backwall will either be connected to normal straight wingwalls or turnback wingwalls. The backwall height is selected based on burying the bottom of the pile cap in the abutment slope a minimum of 700 mm and allowing 1000 mm of headroom between the abutment slope and the bottom of the slab to aid in inspection.

The slab superstructure is connected to the backwall by a series of 25-mm diameter steel dowels made with metal or PVC expansion caps. This connection creates a hinge condition and

allows for future jacking of the slab superstructure in case of settlement. A waterstop is placed between the slab and backwall to control seepage.

For normal crown structures, the pile cap is built level and the backwall height is varied. Typically, one height of U-bar is used for vertical reinforcement, and the embedment into the cap is varied to match the crown of the roadway. On superelevated structures, slope the pile cap to match the superelevation. MDT's minimum cap dimensions are 1000-mm in width and 800 mm in depth. Standard pile embedment into the cap is 500 mm.

Figure 16.3D illustrates a typical section of an end bent with a flat slab.

16.3.14 Sample Design for Flat Slab

16.3.14.1 Sample Calculations

The next several pages present a sample calculation for a haunched, three-span, continuous flat-slab bridge.

16.3.14.2 Typical Details

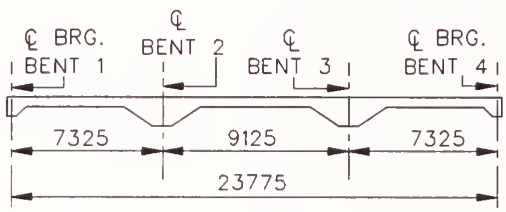
After the sample calculation, Figures 16.3C and 16.3F present details for MDT's typical half longitudinal slab sections and transverse slab sections.

Haunched, 3-Span, Continuous Flat-Slab Bridge

Given: Total Length = 23 775 mm
Roadway Width = 8500 mm
Rail Type: T101

Main-Span Length, L = 9125 mm
End-Span Length, 0.8L = 7325 mm (Section 16.3.1.3)

Sketch:

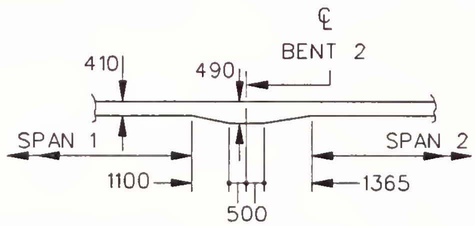


Minimum Slab Thickness = 410 mm (Section 16.3.10)

Haunch Thickness $\cong 1.2t_{min}$ = 490 mm (Section 16.3.1.3)

Length of Haunch $\cong$ 0.15L Main Span: 1365 mm (Section 16.3.1.3)
End Span: 1100 mm

Sketch:



Equivalent Strip Width (LRFD Article 4.6.2.3)

$$E = 250 + 0.42\sqrt{L_1W_1} \quad \text{(LRFD Equation 4.6.2.3 – 1) for single-lane loaded}$$

$$E = 2100 + 0.12\sqrt{L_1W_1} \leq \frac{W}{N_L} \quad \text{(LRFD Equation 4.6.2.3 – 2) for multilanes loaded}$$

W = 8500 + 700 = 9200
N_L = 8500/3600 = 2.36 ≥ 2 (LRFD Article 3.6.1.1.1)
L_1 = 7325 mm (shorter span will control)

$$W_1 = 8500 + 700 = 9200\text{ mm} \leq \left|\begin{array}{l} 9000\text{ mm for single-lane loaded} \\ \\ 18\,000\text{ mm for multilanes loaded.} \end{array}\right.$$

$$E_{single} = 250 + 0.42\sqrt{(7325)(9000)} = 3660\text{ mm}$$

$$E_{multi} = 2100 + 0.12\sqrt{(7325)(9000)} = 3085\text{ mm}$$

$$\frac{W}{N_L} = \frac{9200}{2} = 4600\text{ mm}$$

Therefore, E = 3085 mm (Interior Strip)

Equivalent Strip at Edge of Slab (LRFD Articles 4.6.2.1.4b and 9.7.1.4)

$$E = 350 + 300 + \frac{1}{2}(3085)$$

$$= 2192.5\text{ mm, but not exceeding } 3085 \text{ or } 1800.$$

$$\therefore \text{use } E = 1800\text{ mm } (\text{Edge Beam})$$

Dead Load

Interior Strip: $\left(\frac{3085\text{ mm}}{1000}\right)\left(\frac{410\text{ mm}}{1000}\right)(2400\text{ kg/m}^3) = 3036\text{ kg/m}$

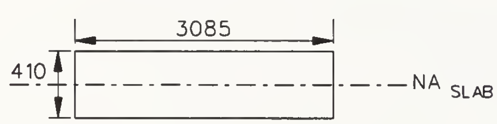
$= 29.8\text{ kN/m}$

Edge Beam: $\left(\frac{1800\text{ mm}}{1000}\right)\left(\frac{410\text{ mm}}{1000}\right)(2400\text{ kg/m}^3) = 984\text{ kg/m}$

$= 9.7\text{ kN/m}$

Modeling Section Properties for BTBEAM

Slab Section

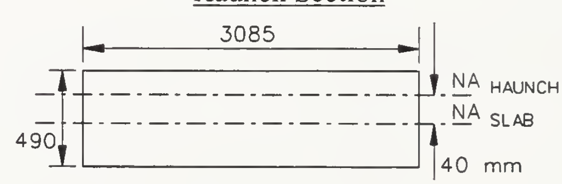


$$I = \frac{bh^3}{12}$$

$$= \frac{(3085)(410)^3}{12}$$

$$= 1.77 \times 10^{10}\text{ mm}^4$$

Haunch Section



$$I = \frac{bh^3}{12} + Ad^2$$

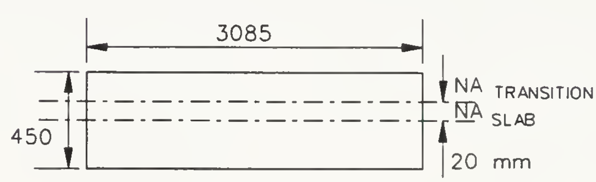
$$= \frac{(3085)(490)^3}{12} + 490(3085)(40)^2$$

$$= 3.02 \times 10^{10} + 2.42 \times 10^9$$

$$= 3.27 \times 10^{10}$$

Slab Transition Section

$$t_{avg} = \frac{410 + 490}{2} = 450$$

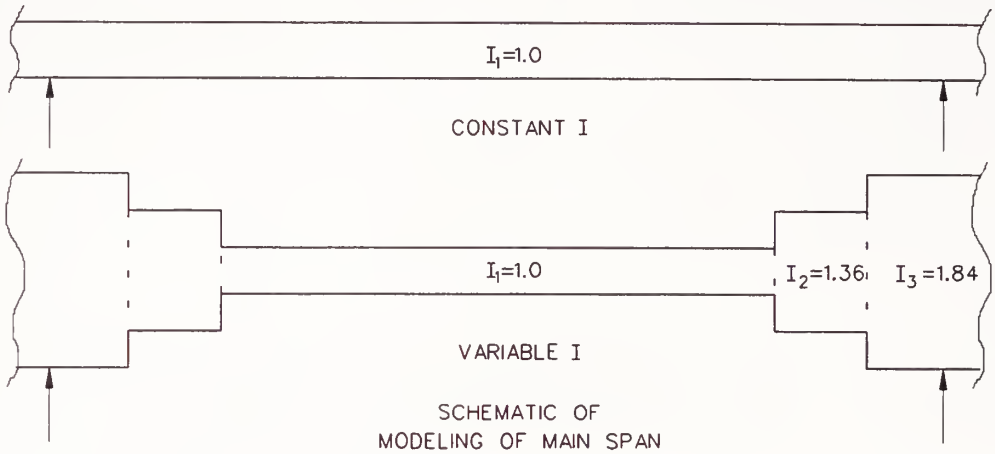


$$I = 2.34 \times 10^{10} + 5.56 \times 10^8$$

$$= 2.40 \times 10^{10}$$

BTBEAM Model

$$I_1 = I_{slab} / I_{slab} = 1.0$$
$$I_2 = I_{tansition} / I_{slab} = 1.36$$
$$I_3 = I_{haunch} / I_{slab} = 1.84$$



Results of BTBEAM Analysis of Interior Strip

Strength I Moments	Constant I	Variable I	Refined
End span	876 kN • m	847 kN • m	838 kN • m
	-893 kN • m	-988 kN • m	-1013 kN • m
Main span	880 kN • m	818 kN • m	800 kN • m

Discussion: The "Refined" analysis included the moment of inertia (I) at several sections along the haunch in the BTBEAM model. The results of the three methods of analysis indicate that the Constant "I" model grossly underestimates the negative moment at the supports compared to the Refined model. The results of the Variable "I" model and the Refined model are within 5% of each other, which is an acceptable tolerance for this design. Therefore, the Variable I method is recommended.

Design of Rectangular Flexural Sections

Design the typical cross sections at the point of maximum positive moment (mid-center span) and maximum negative moment (interior support) using LRFD Article 5.7.3 as appropriate.

$$\sum \gamma_i M_i = 847 \text{ kN} \bullet \text{m on } 3085 - \text{mm Interior Strip (maximum positive)}$$

or

$$\frac{847 \text{ kN} \bullet \text{m}}{3.085 \text{ m}} = 275 \text{ kN} \bullet \text{m/m}$$

$$\sum \gamma_i M_i = -988 \text{ kN} \bullet \text{m on } 3085-\text{mm Interior Strip (maximum negative)}$$

or

$$\frac{-988 \text{ kN} \bullet \text{m}}{3.085 \text{ m}} = 321 \text{ kN} \bullet \text{m/m}$$

$$M_r = \varphi M_n \qquad \text{(LRFD Equation 5.7.3.2.1-1)}$$

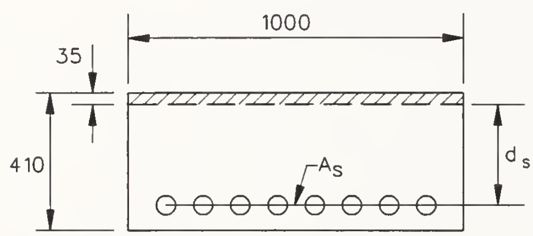
$$M_n = A_s f_y \left(d_s - \frac{a}{2} \right) \qquad \text{(LRFD Equation 5.7.3.2.2-1)}$$

$$a = \beta_1 C \qquad \text{(LRFD Article 5.7.2.2)}$$

$$c = \frac{A_s f_y}{0.85 f'_c \beta_1 b} \qquad \text{(LRFD Equation 5.7.3.1.2-4)}$$

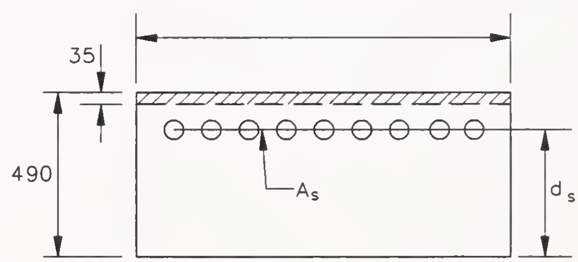
Cross section

MID SPAN (MAX. POSITIVE MOMENT)



A_s = 2241 mm^2 (REQUIRED)
OR #22 BARS @ 170-mm CTRS

INTERMEDIATE SUPPORT
(MAX. NEGATIVE MOMENT)



A_s = 2754 mm^2 (REQUIRED)
OR #25 BARS @ 185-mm CTRS

Transverse Distribution Reinforcement

Bottom of Slab (LRFD Article 5.14.4.1):

$$\frac{1750}{\sqrt{L}} \le 50\% \qquad \text{(Maximum spacing (Article 5.10.3.2) = 450 mm)}$$

$$L = 7325 \text{ mm} \qquad \text{(Shorter span will control)}$$

$$\frac{1750}{\sqrt{7325}} = 20.5\% \qquad (A_s - 2241 \text{ mm}^2 (0.205) = 459 \text{ mm}^2\text{, or \#16s @ 450-mm centers})$$

Shrinkage and Temperature Reinforcement

Top and Bottom of Slab (LRFD Article 5.10.8):

$A_s \geq 0.75\ A_g / f_y$

$$\geq \frac{0.75\,(1000)(410)}{420} = 732\ \frac{mm^2}{m}$$ (Equally distributed on top and bottom faces of slab in each direction.)

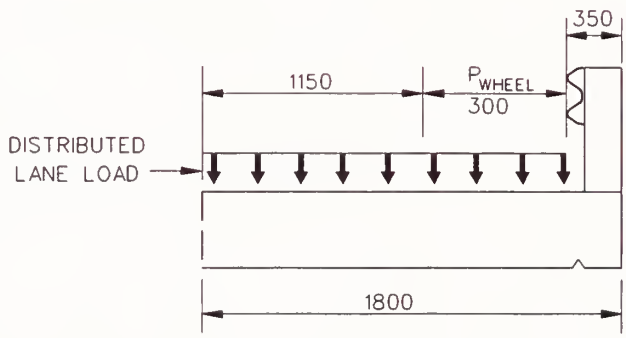
Maximum spacing = 3(410 mm) = 1230 mm or 450 mm

A_s = 366 mm^2 or #16s @ 450-mm centers (Top of slab, transverse)

Note: Shrinkage and temperature steel requirements, for top longitudinal, bottom longitudinal and bottom transverse, already satisfied by flexural and distribution requirements checked previously.

Analysis of Edge Beam

Sketch:



The BTBEAM live load analysis results are for an entire lane. Reduce the truck load results by ½ to get results for one wheel line, and reduce the redistributed lane load by 1800/3600 to determine the load on the notional edge beam.

Design of Rectangular Flexural Sections

BTBEAM analysis results for the edge beam using the Variable I method:

Strength I Moments	End span: + 440 kN • m	
	- 432 kN • m	(on 1800-mm strip)
	Main span: + 424 kN • m	

$$\sum \gamma_i M_i = \frac{440 \text{ kN} \bullet \text{m}}{1.800 \text{ m}} = 244 \text{ kN} \bullet \text{m/m} \quad \text{(maximum positive)}$$

$$\sum \gamma_i M_i = \frac{-432 \text{ kN} \bullet \text{m}}{1.800 \text{ m}} = 240 \text{ kN} \bullet \text{m/m} \quad \text{(maximum negative)}$$

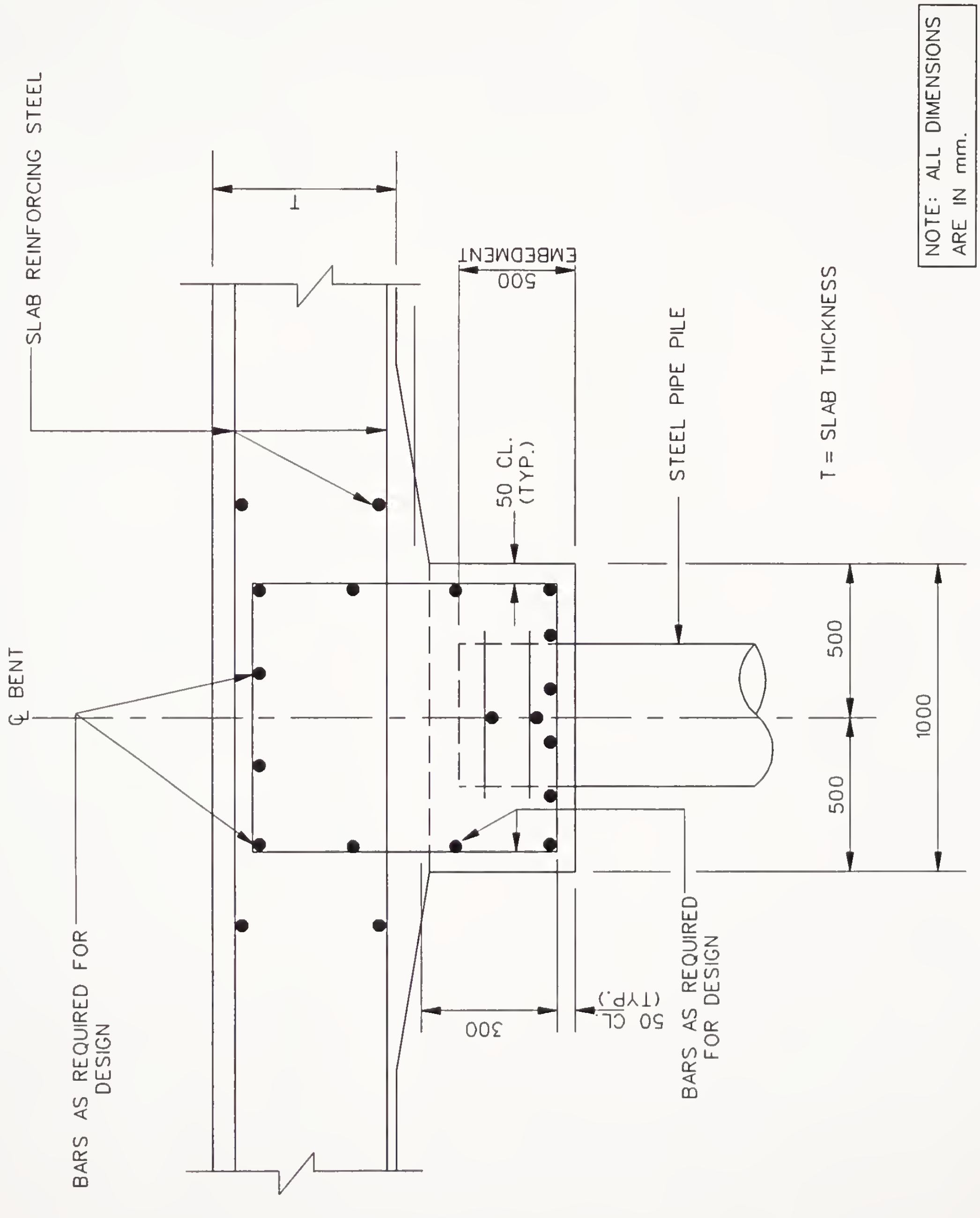
See Figure 16.3E for typical cross section.

$A_s = 3627 \text{ mm}^2$ (required for maximum positive moment)

or 8 #25s top and bottom.

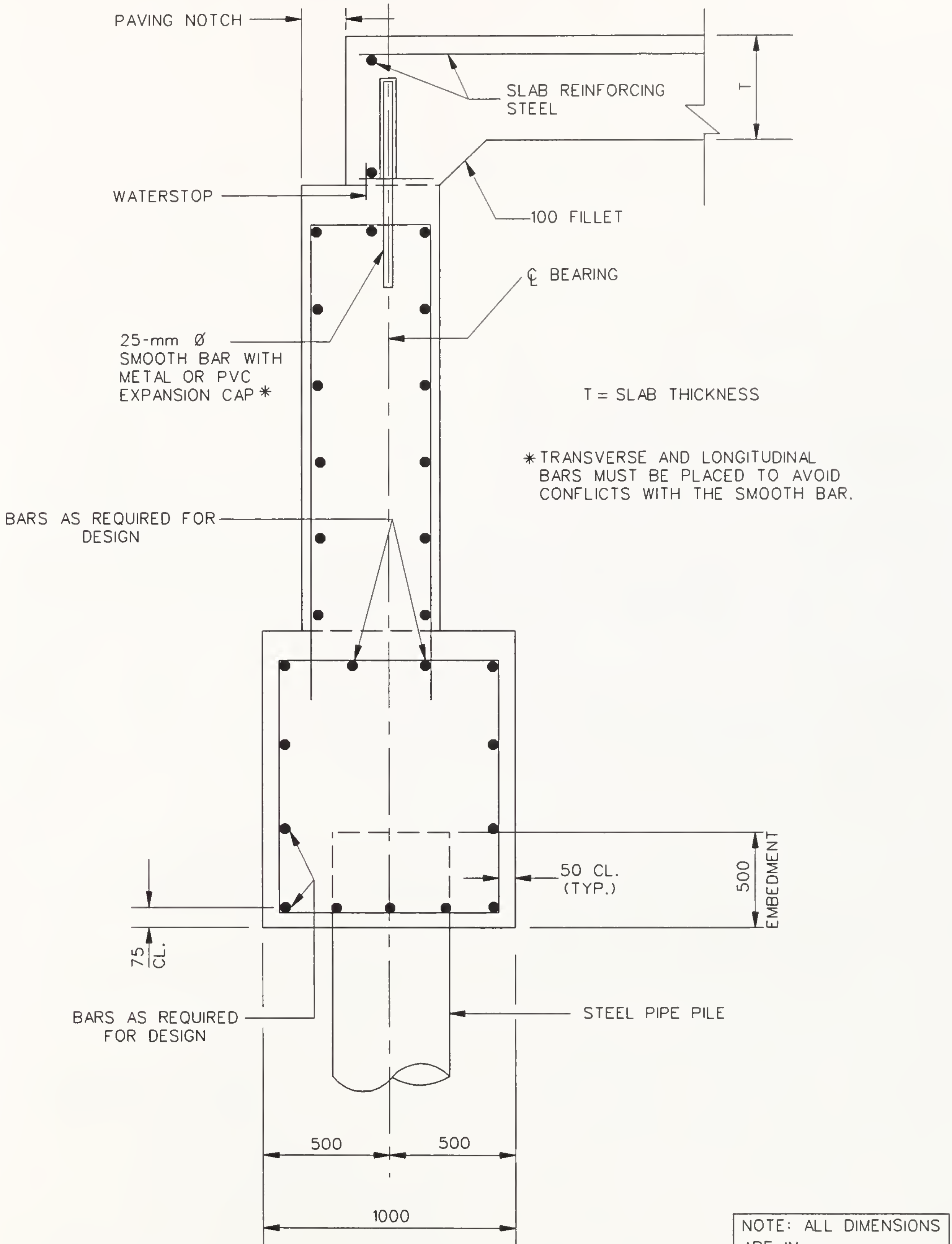
Shear Design of Edge Beam

For a constant-depth slab and integrated edge beam, shear reinforcing will not be necessary per Article 5.14.4.1. In special circumstances where a thickened edge beam is provided, shear should be investigated as set forth in Article 5.8.



INTEGRAL CAPS AT INTERMEDIATE BENTS OF FLAT SLABS

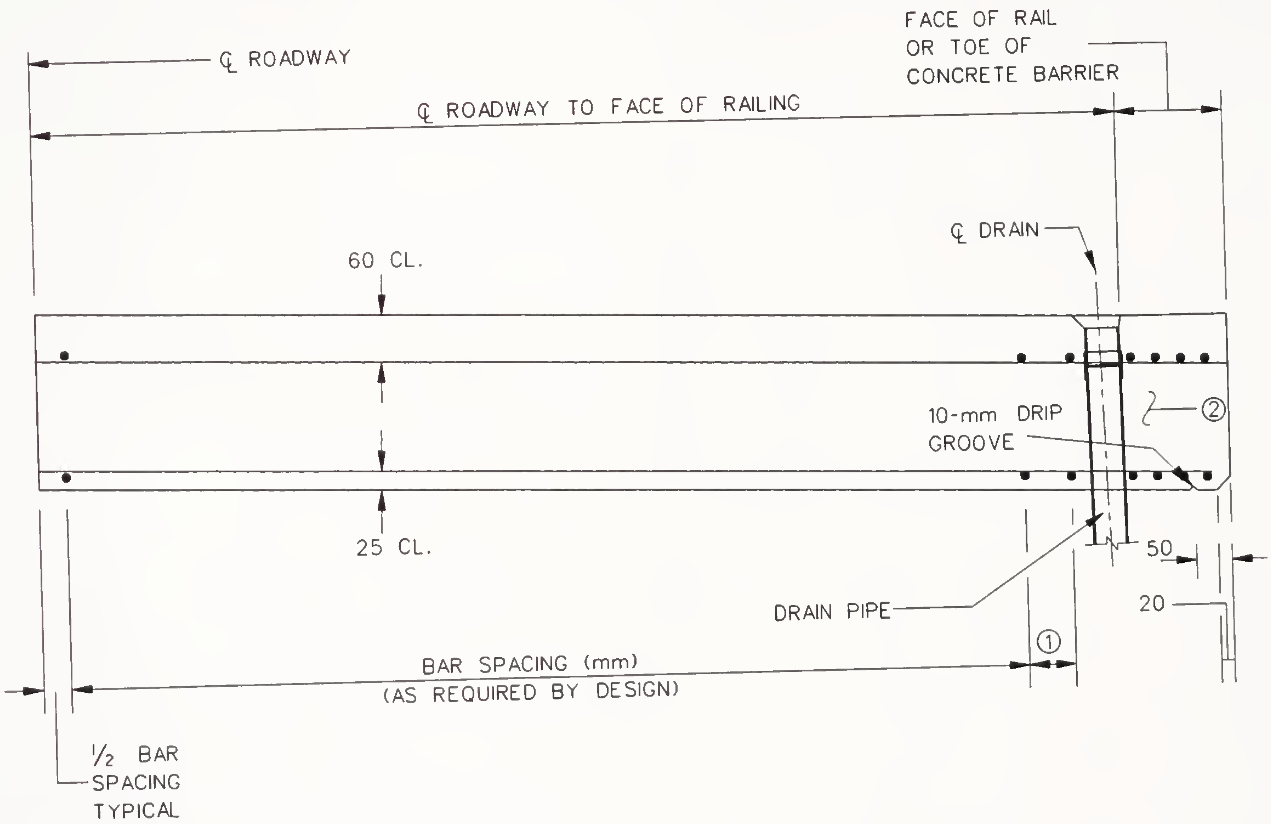
Figure 16.3C



END BENTS OF FLAT SLABS
(Section Through End Bent)

Figure 16.3D

NOTE: ENTIRE TYPICAL SECTION TO BE DETAILED ON PLANS.



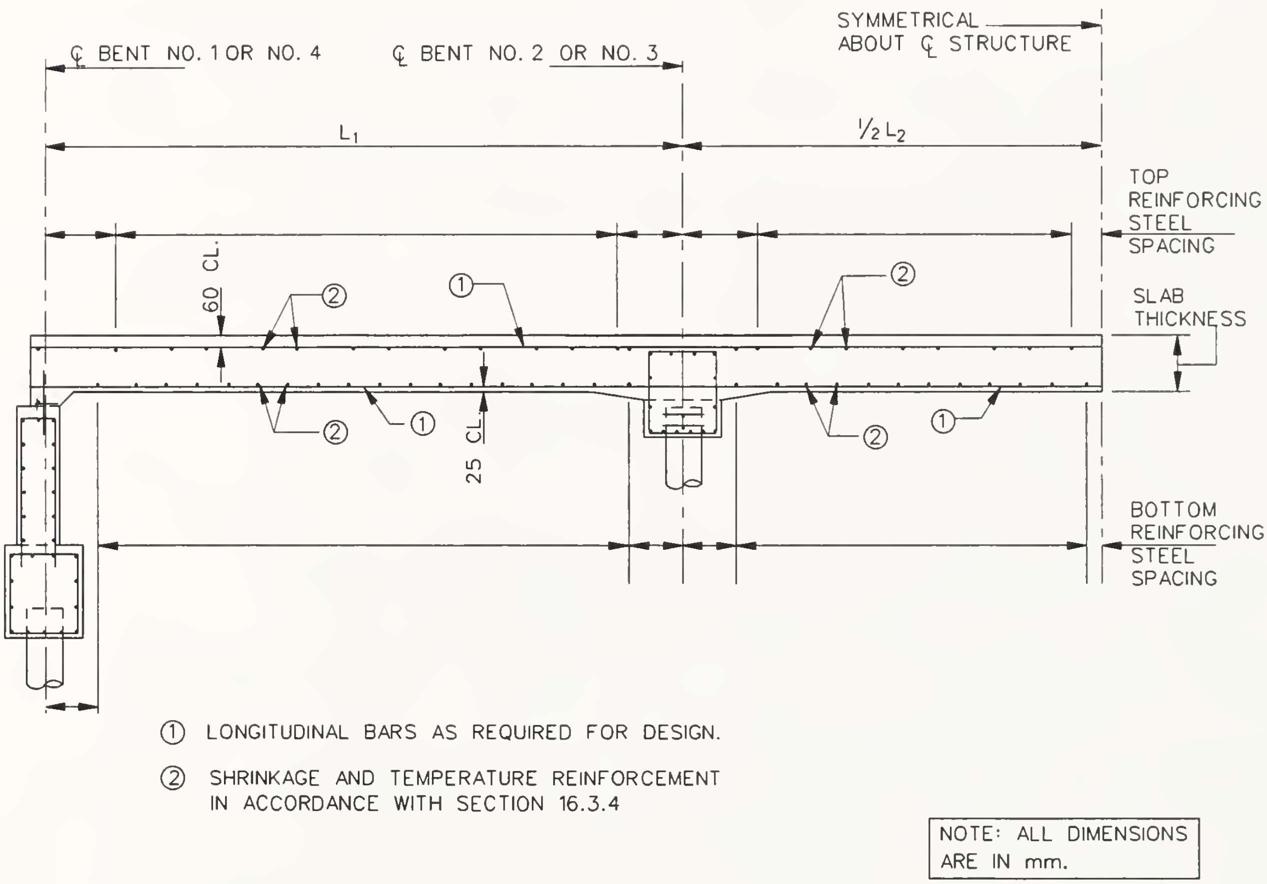
① BAR SPACING AND NUMBER OF SPACES TO BE DETERMINED TO FACILITATE A CONSTANT BAR SPACING IN REMAINDER OF SLAB.

② DESIGN EDGE BEAM IN ACCORDANCE WITH ARTICLES IN THE LRFD SPECIFICATIONS, BUT USE AS A MINIMUM THE SAME AREA OF STEEL PER METER AS IN SLAB.

NOTE: ALL DIMENSIONS ARE IN mm.

EDGE BEAM DETAIL, DECK SECTION OF FLAT SLABS
(Typical Half Section)

Figure 16.3E



INTEGRAL CAPS AT FLAT SLABS
(Half Longitudinal Section)

Figure 16.3F

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(Continued)

Chapter Seventeen

PRESTRESSED CONCRETE SUPERSTRUCTURES

Section 5 of the **LRFD Bridge Design Specifications** unifies the design provisions for concrete reinforced with steel reinforcing bars and/or prestressing tendons. Chapter Seventeen presents MDT supplementary information specifically for fully prestressed concrete components. Chapter Sixteen discusses reinforced concrete components.

17.1 GENERAL

17.1.1 Definitions

Reference: LRFD Article 5.2

The following definitions apply:

1. Prestressing. Prestressing is the process of inducing stresses and deformations into a component prior to the application of the external loads. In the context of this section, high-strength steel is used to induce compressive stresses and deformations into a concrete component.
2. Pretensioning. Pretensioning is the process in which a concrete component is prestressed by releasing into it the force of steel tendons that are tensioned prior to concrete being cast around the tendons.
3. Posttensioning. Posttensioning is the process in which a concrete component is prestressed by tensioning steel tendons that are inserted into ducts cast through the component.
4. Partial Prestressing. Partial prestressing is the process of prestressing in which the concrete component is allowed to experience service-load tension; i.e., the concrete component resists loads through prestressed tendons and mild reinforcement.

17.1.2 Practices and Responsibilities

17.1.2.1 MDT Practices

MDT's prestressed concrete designs are typically limited to precast, fully prestressed girders. Therefore, the design provisions discussed herein address solely precast, prestressed girders.

Posttensioning or partial prestressing shall be used only with the approval of the Bridge Design Engineer.

17.1.2.2 Responsibilities

MDT's preliminary design process must ensure that the chosen standard girder can achieve the specified resistance. In other words, can the prestressing force be achieved within the cross section and the allowable stresses met. For MDT design applications, it is assumed that the girder can be fabricated in a single day with a released strength of 41.5 MPa.

The MDT preliminary design results in a specified ultimate moment capacity, termed the nominal resistance in the LRFD Specifications, and the allowable top and bottom stresses for the chosen girder section and girder spacing.

The fabricator is responsible for the final design of the prestressed concrete girder. Shop drawings and a complete set of final design calculations are submitted by the fabricator for Bridge Bureau approval.

The fabricator or general contractor is responsible for investigating stresses in the components during proposed handling, transportation and erection.

17.2 MATERIALS

17.2.1 Concrete

Reference: LRFD Article 5.4.2.1

The typical specified 28-day compressive strength for precast, prestressed girders is 48 MPa. The specified strength may be increased to 52 MPa with the approval of the Bridge Area Engineer.

17.2.2 Prestressing Tendons

17.2.2.1 Material Properties

Reference: LRFD Article 5.4.4.1

The prestressing tendons typically used for prestressed girders in Montana are low-relaxation, seven-wire strands conforming to AASHTO M203 (ASTM A416), Grade 1860. The tensile strength, f_{pu}, and yield strength, f_{py}, of this steel are 1860 MPa and 1675 MPa, respectively.

17.2.2.2 Geometric Properties

References: LRFD Articles 5.4.4.1 and 5.11.4.3

The following applies:

1. Diameter. Seven-wire strand of 12.7-mm diameter, with an area of 98.77 mm^2, are the typical tendons used in Montana.

2. Number. An even number of strands must be specified.

3. Spacing. The minimum spacing of seven-wire strand tendons shall be 50 mm center to center.

4. Trajectory. For simplicity of fabrication, straight trajectories are preferred. However, harped trajectories are more typical. Harped trajectories help to control stresses and camber, and they contribute to shear resistance. An alternative to harping strands is debonding or shielding strands. The LRFD limit of a maximum of 25% of the strands debonded shall be observed.

17.3 BRIDGE DETAILS

17.3.1 Continuity

Reference: LRFD Article 5.14.1.2.7

17.3.1.1 General

Multiple-span, prestressed concrete girder bridges consist of simple span girders supporting continuous cast-in-place, reinforced concrete decks. In some cases, the prestressed concrete girders are made continuous after erection to resist live load as continuous girders. In these cases, the stiffness of the cast-in-place diaphragms encasing the ends of the precast girders over the piers is neglected.

17.3.1.2 Slab Reinforcement

Reference: LRFD Articles 5.11.1.2.3 and 5.14.1.2.7b

If a bridge is designed so that the girders and slab act continuously over the bent, the longitudinal reinforcement in the slab over an internal pier shall be anchored in zones that are crack-free; i.e., in compression, at strength limit states. This means that the steel used to resist the negative moment must extend past the point of contraflexure. The embedment length into the compression zone shall satisfy LRFD Article 5.11.1.2.3. Further, the terminating bars shall be staggered.

This treatment should not be confused with the routine placement of S5 and S6 Bars (see the **MDT Standard Bridge Details** MSL-5 and MSL-6) as additional reinforcing steel in the slab over bents where only the slab is continuous.

17.3.2 Diaphragms

Reference: LRFD Article 5.13.2.2

17.3.2.1 General

Standard intermediate and end diaphragm details, as shown on **MDT Standard Bridge Details** MSL-5 and MSL-6, shall be used on all prestressed girder superstructures, where applicable.

17.3.2.2 Intermediate Diaphragms

Intermediate diaphragms shall be spaced as shown on Standard Drawings MB-1, MB-A, MB-4, MM-72 and MMT-28. All intermediate diaphragms shall be placed normal to the centerline of the girders.

17.3.2.3 End Diaphragms

End diaphragms shall be provided at all intermediate supports and abutments where expansion joints are located.

End diaphragms at intermediate supports shall be placed parallel to the skew of the abutment.

17.3.2.4 Diaphragms for Girders Made Continuous for Live Load

There are several possible treatments at the girder end to structurally engage the diaphragm and attain design continuity. Among the possible treatments are the following:

1. Serrate the girder end and extend the mild reinforcing steel for shear transfer to the diaphragm. Extend a portion of the prestressing strands from the girder bottom for positive moment capacity over the support caused by live loading in remote spans. The girder is temporarily supported on hardwood blocks until the diaphragm concrete has cured.

2. Set girder ends on thin elastomeric pads directly on the cap. Extend a portion of the prestressing strands from the girder bottom for positive moment capacity over the

support caused by live loading in remote spans. Place the diaphragm concrete around the girder ends. The diaphragm is the width of the cap below it.

Obtain the approval of the Bridge Area Engineer for the methods to be used prior to proceeding with design.

17.4 STANDARD GIRDERS

17.4.1 General

MDT has available for bridge design a series of prestressed, precast concrete girder sections. The sections to be used for design arc those historically produced by local fabricators. Fabricators are allowed to supply girders with similar section properties provided that the girder design meets contract requirements.

General information on the available girder types and typical span ranges is presented in Chapter 13 of this **Manual**. More specific design information for prestressed girder bridges is presented in this Chapter.

One standard girder cross section shall be used exclusively in a structure, unless unusual circumstances dictate a need to vary girder section depth within a structure. An example might be a localized vertical clearance problem. Any intended use of different depth girders within a single structure must be approved by the Bridge Area Engineer in writing before the layout is finalized.

Girder lengths are usually rounded to the nearest 0.5-m increment at the conclusion of the bridge length calculations.

17.4.2 Prestressed I-Girder Section

Unless otherwise approved, simple span lengths for the various standard girders shall not exceed the centerline-to-centerline-bearing lengths in Figure 17.4A.

Girder Type	Span Length Limit (m)
Type 1	17
Type MT-28	23
Type A	26
Type IV	35
Type M-72	45

STANDARD GIRDER LENGTHS
Figure 17.4A

Slightly greater span lengths may be used for girders made continuous for live load. For I-Girder section properties, see Figurc 17.4B

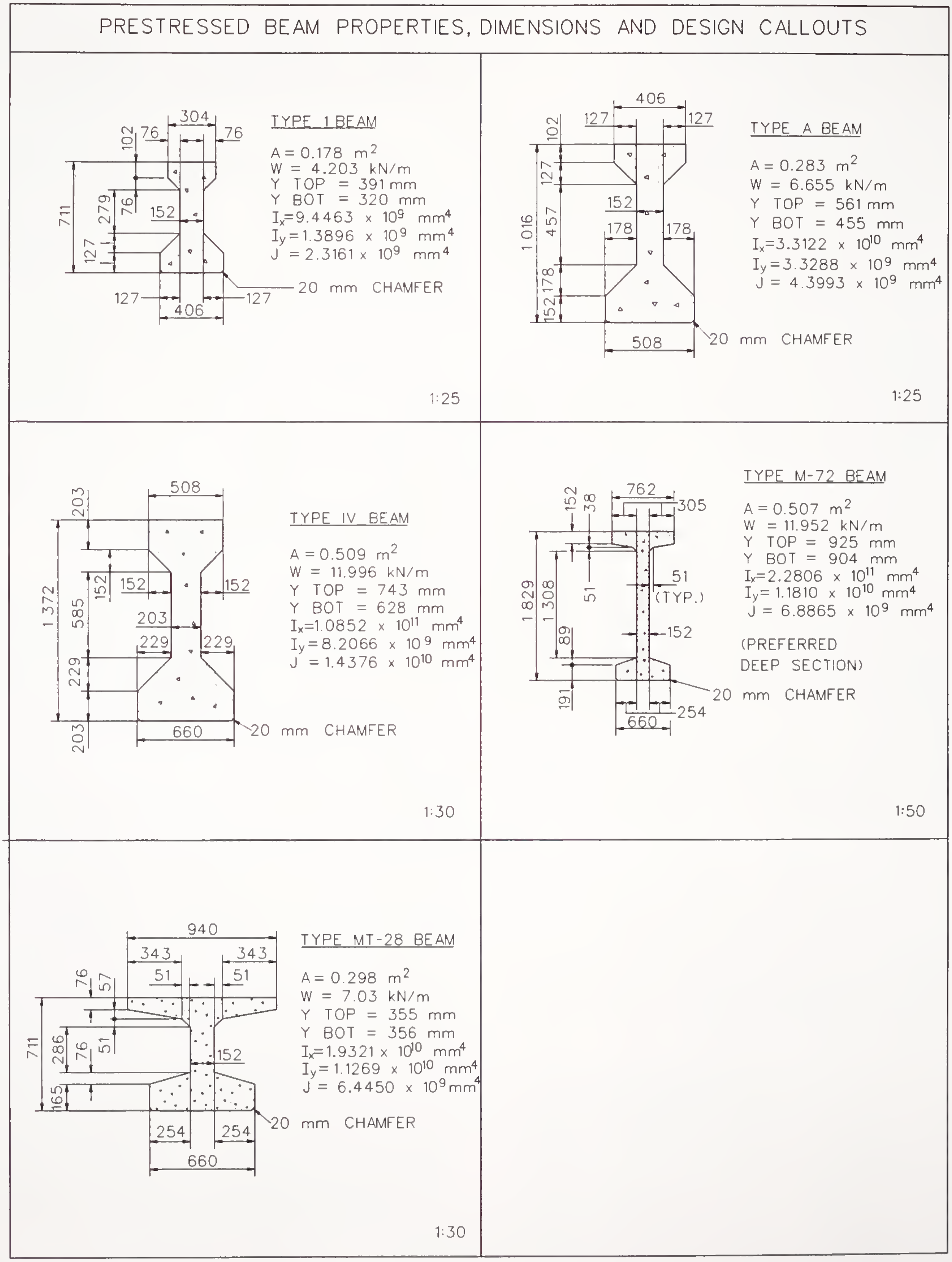
17.4.3 Joined Prestressed Precast Girder Sections

Also available are two types of joined prestressed, precast girder sections. These are bulb-T girders and tri-deck girders. These secions are placed side-by-side forming an instant roadway making them a good choicc for remote locations where concrete availability is a problem or where reducing the duration of a road closurc can save the cost of a detour. The jointed precast sections can be fabricated in a variety of depths and widths that allow these girders to be customized over a wide range of span lengths and roadway widths.

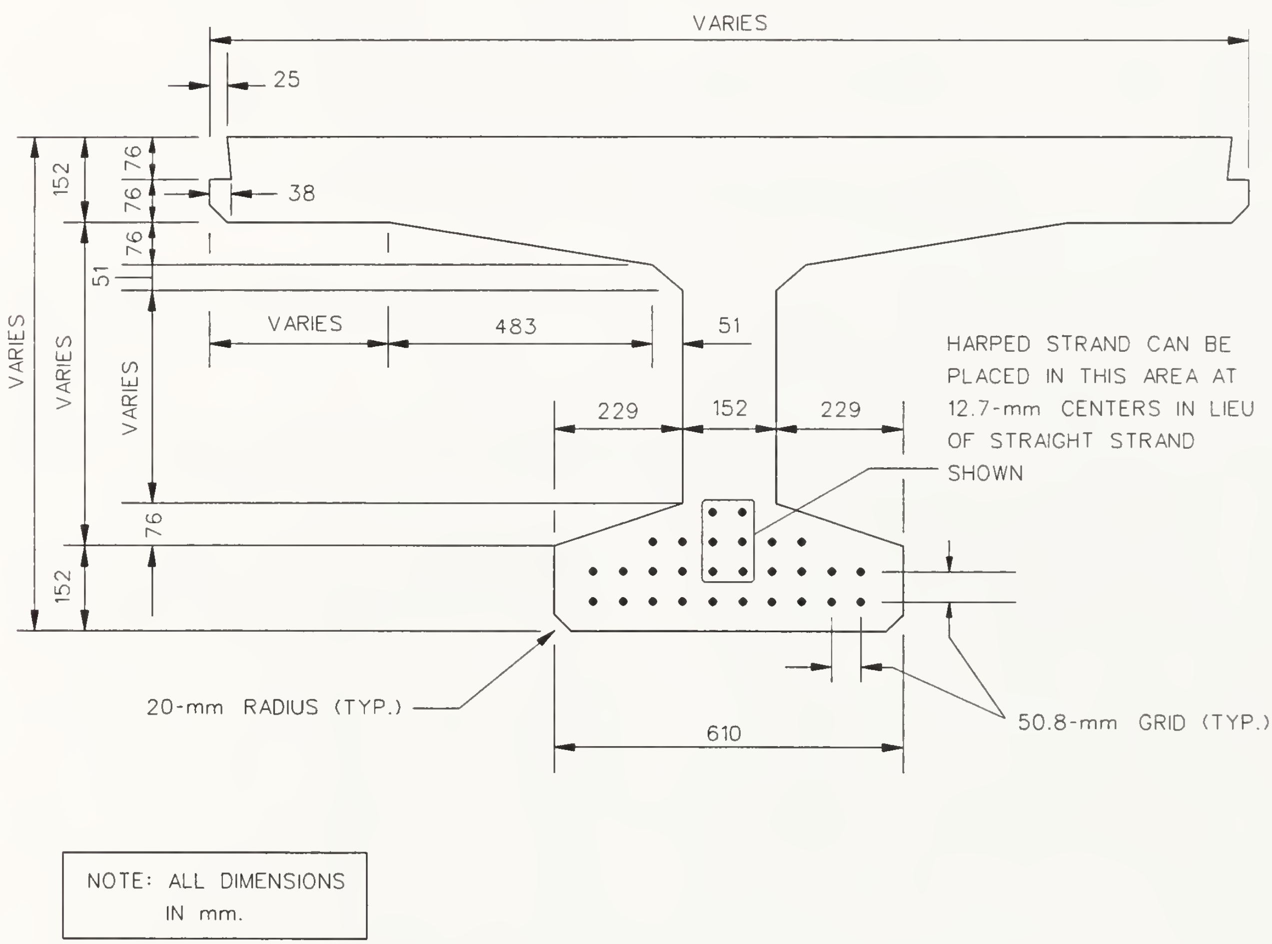
Figure 17.4C identifies the general shape of a bulb-T girder. Because of the wide variety of possible dimensions, no section properties have been included in this **Manual**. The designer should consult with local fabricators for the latest information to use in design calculations.

Bulb-T girders can be fabricated in a range of depths from 889 mm to 1422 mm, and the top flange can vary in width between 1219 mm and 2438 mm. The bottom flange dimensions and the web thickness are constant for all sections. This girder type allows for economical spans from about 14 m to about 40 m in length.

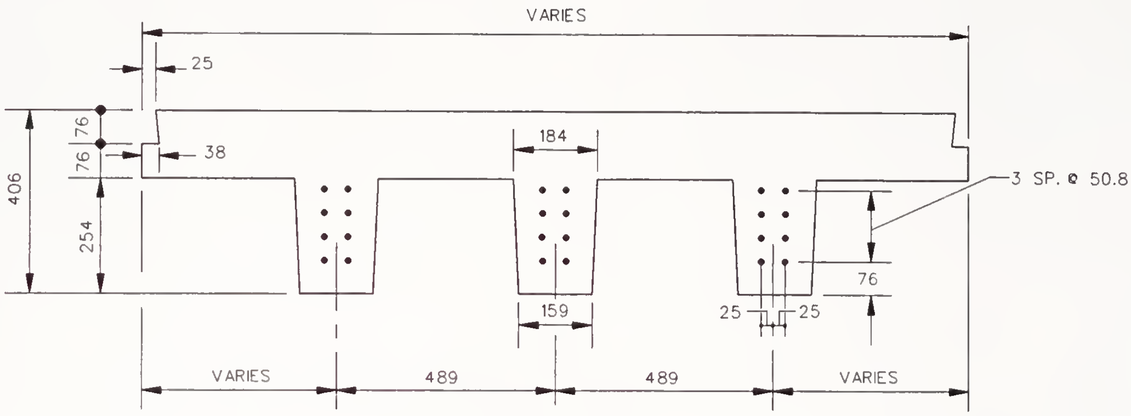
Figure 17.4D identifies the general shape of a tri-deck girder. Tri-deck girders come in two depths — 406 mm or 685 mm. The range of possible top flange widths is the same for both depths because they can be varied from 1212 mm to 1822 mm. This girder type allows for economical spans from approximately 10 m to 22 m in length. Because of their shallow depth, tri-deck girders are often considered as an option to cast-in-place flat slab bridges.



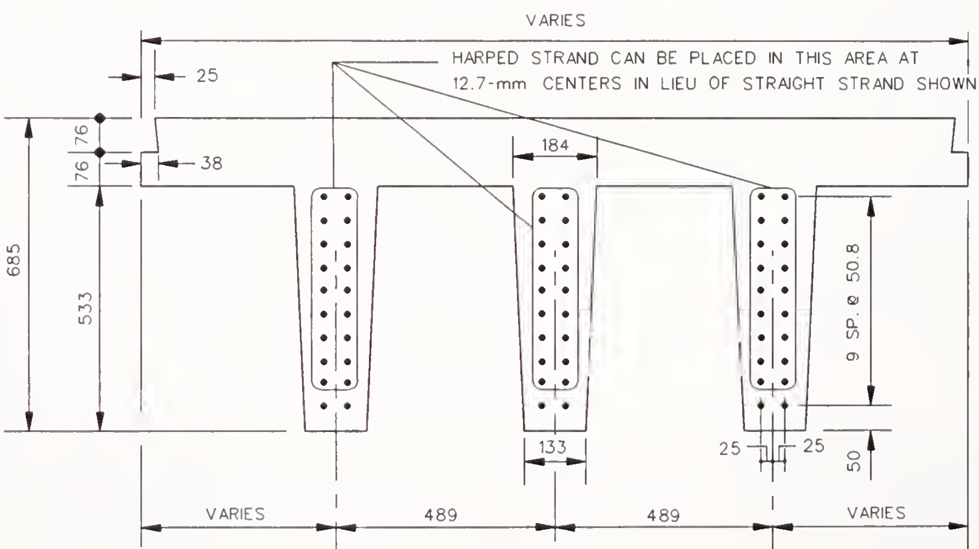
PROPERTIES OF STANDARD GIRDER SECTIONS
Figure 17.4B



BULB-T
Figure 17.4C



SHALLOW



DEEP

NOTE: ALL DIMENSIONS IN mm.

TRI-DECK
Figure 17.4D

17.5 GIRDER DESIGN

17.5.1 General

In general, bridge cross sections shall be selected which yield the minimum number of girders.

17.5.2 Prestressed Losses

Reference: LRFD Article 5.9.5

Time-dependent prestressed losses shall be taken as tabularized in LRFD Table 5.9.5.3-1 or as calculated as itemized values by the MDT computer program, Prestressed Beam Design.

17.5.3 Flexural Resistance

17.5.3.1 General

References: LRFD Articles 5.7.2.2, 5.7.3.1.1 and 5.7.3.2.2

The general LRFD equation for the nominal, flexural resistance of flanged concrete sections when applied to fully prestressed concrete sections reduces to the following:

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85f'_c\,(b - b_w)\beta_1 h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

where:

A_{ps} = area of prestressing steel

f_{ps} = average stress in prestressing steel at nominal flexural resistance specified in LRFD Article 5.7.3.1.1

d_p = distance from extreme compression fiber to the centroid of prestressing tendons

a = depth of the equivalent stress block

b = width of compression face of the member

b_w = web width

β_1 = stress block factor specified in LRFD Article 5.7.2.2

h_f = compression flange depth

17.5.3.2 Example

The end of Section 17.5 presents a typical flexural design of a simple span, Type M72 prestressed concrete girder with a continuous bridge deck.

17.5.4 Shear Resistance

Reference: LRFD Article 5.8

The drawings for the MDT Standard sections were developed by using an envelope of "worst-case" loadings for simple spans. In general, shear does not need to be checked on a project-specific basis if using standard girder sections. However, if the designer has a situation where shear needs to be investigated (e.g., girders made continuous for live load), the following discussion may be helpful.

17.5.4.1 Shear Resistance of Concrete

In the LRFD Specifications, the shear resistance of the concrete, V_c, shall be taken as the lesser of V_{ci} or V_{cw}:

$$V_{ci} = 0.6\sqrt{f'_c}\,b'd + V_d + \frac{V_i M_{cr}}{M_{max}}$$

but need not be less than:

$$1.7\sqrt{f'_c}\,b'd$$

and

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3f_{pc})b'd + V_p$$

where:

b' = width of the web of a flanged member

d = distance from extreme compressive fiber to centroid of the prestressing force

V_d = shear force at section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

M_{cr} = moment causing flexural cracking at section due to externally applied loads

M_{max} = maximum factored moment at section due to externally applied load

f_{pc} = compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads

V_p = vertical component of effective prestress force at the section

The shear resistance of the concrete thus calculated shall be used in LRFD Equation 5.8.3.3-1.

17.5.4.2 Shear Resistance of Steel

Reference: LRFD Article 5.8.3.3

The general LRFD Equation for the shear resistance of transverse steel reinforcement, V_s, can be simplified for use with the traditional value of concrete shear resistance and vertical stirrups as follows:

$$V_s = \frac{A_s f_y d}{s}$$

17.5.5 Erection Plan Drawing Notes

Erection plan drawings have several entries that pertain to the design of the prestressed girders. A note that sets the maximum f_c' (compressive strength of concrete at 28 days) for design to 48 MPa is placed on the drawing.

A tabulation of design stresses at extreme fibers without prestress is made for girder dead load stresses and total dead load plus live load design stresses. The required factored moment in kN-m is also shown.

A typical beam-design-stresses table is shown in the example at the end of Section 17.5.

Prestressed Concrete Beam Example

Given: Type M-72 Girder:

A	=	$0.507\ m^2$
W	=	11.952 kN/m
y_{top}	=	925 mm
y_{bottom}	=	904 mm
I	=	$2.2806 \times 10^{11}\ mm^4$

Roadway Width	=	12 m (face to face of curbs)
Span Length	=	35 m
Girder Spacing	=	2650 mm (5-girder cross section)
Slab Thickness	=	200 mm
Haunch Depth	=	220 mm
Wearing Surface	=	35 mm
f'_c Girder	=	48 MPa (41.5 MPa @ transfer) (Section 17.1.2.2)
f'_c Slab	=	31 MPa

Interior Girder Design
HL-93 Live Load

Loads:

Dead Load Moments:

Girder: $M = \frac{WL^2}{8} = \left(11.952\frac{kN}{m}\right)\frac{(35\ m)^2}{8} = 1830\ kN \bullet m \qquad \gamma = 2400\frac{kg}{m^3}$

Slab: $(2.65\ m)\left(\frac{200\ m}{1000}\right)\left(23.537\frac{kN}{m^3}\right)\left(\frac{(35\ m)^2}{8}\right) = 1910\ kN \bullet m$

Fillet: $\left(\frac{762\ mm}{1000}\right)\left(\frac{20\ mm}{1000}\right)(23.537)\left(\frac{35^2}{8}\right) = 55\ kN \bullet m$

Barrier : $2\left(\frac{5.187\ kN/m}{5\ girders}\right)\left(\frac{(35\ m)^2}{8}\right) = 318\ kN \bullet m$

Diaphragm: $M = P\frac{L}{3} = (24.55\ kN)\frac{35}{3} = 286\ kN \bullet m$

Future Wearing Surface: $\left(0.5\frac{kN}{m^2}\right)\left(\frac{12\ m}{5\ girders}\right)\left(\frac{(35\ m)^2}{8}\right) = 184\ kN \bullet m$

Live Load Moment:

Interior girder, multiple lanes:

$$g = 0.075 + \left(\frac{S}{2900}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{k_g}{Lt_S^3}\right)^{0.1} \quad \text{(LRFD Table 4.6.2.2.2b-1)}$$

where: $k_g = n(I + Ae_g^2)$

$$n = \frac{33\,200 \text{ MPa}}{26\,700 \text{ MPa}} = 1.243$$

$I = 2.2806 \times 10^{11}\ mm^4$ (Figure 17.4B)

$A = 0.507\ m^2$

$$e_g = 925 \text{ mm} + 20 \text{ mm} + \frac{165}{2} = 1028 \text{ mm}$$

$$k_g = 1.24\,[2.28 \times 10^{11} + 0.507\,(1028)^2] = 9.46 \times 10^{11} \text{ mm}^4$$

$$g = 0.075 + \left[\frac{2650}{2900}\right]^{0.6}\left[\frac{2650}{35\,000}\right]^{0.2}\left[\frac{9.46 \times 10^{11}}{(35\,000)(165)^3}\right]^{0.1}$$

$$= 0.075 + (0.947)(0.597)(1.197)$$

$$g = 0.075 + 0.676 = 0.752$$

IM = 0.33 for truck and tandem

M_{LL+IM} = (4691.5 kN•m) 0.752 @ mid-span (LL + IM moment from BTBEAM)
= 3528 kN•m

Section Properties:

$$\frac{E_{slab}}{E_{girder}} = \frac{26\,700}{33\,200} = 0.804$$

$$t_{effective} = 200 - 35 = 165 \text{ mm}$$

$$\frac{1}{4}(L) = \frac{1}{4}(35 \text{ m}) = 8.75 \text{ m}$$

$$12(t_{effective}) + \frac{1}{2}(762) = 2361 \text{ mm}$$ (governing effective slab width)

$S = 2560$ mm

Location of composite section NA:

Area:

$$NA = \frac{\Sigma Ay}{\Sigma A}$$

		ΣA		y		ΣAy
0.804 (2361 mm)(165 mm)	=	313 210.30	•	1931.5	=	604 965 694.5
0.804 (762 mm) (20 mm)	=	12 252.96	•	1839	=	22 533 193.4
		507 000.00	•	904	=	458 328 000.0
		832 463.26				1 085 826 887.9

NA = 1304.4 from bottom = y_B

$y_T = (1829 + 20 + 165) - y_B = 709.6$ mm

Composite Section Modulus, I_{comp}: $I = I_0 + Ad^2$

Slab: $$\frac{0.804(2361)(165)^3}{12} + 313\,210.3\,(627.1)^2$$

$= 710\,595\,777.4 + 123\,171\,331\,732$
$= 123\,881\,927\,509$

Haunch: $$\frac{(0.804)(762)(20)^3}{12} + 12\,252.96\,(534.6)^2$$

$= 408\,432 + 3\,501\,861\,169.6$
$= 3\,502\,269\,601.6$

Girder: $2.2806 \times 10^{11} + (507\,000)(400.4)^2$
$= 2.2806 \times 10^{11} + 81\,282\,321\,120$
$= 309\,342\,321\,120 \text{ mm}^4$

I_{comp}: $= 4.367 \times 10^{11} \text{ mm}^4$

Stresses: $$\sigma = \frac{MC}{I}$$

DC beam: $$\left[\frac{(1830\,\text{kN}\bullet\text{m})(1000)(1000)}{2.2806 \times 10^{11}\,\text{mm}^4}\right] \begin{array}{lll} \times & 925\text{ mm} & = 7.422\text{ MPa (Top)} \\ \times & 904\text{ mm} & = 7.254\text{ MPa (Bottom)} \end{array}$$

Slab, fillet & diaphragm: $$\left[\frac{(2251)(1000)(1000)}{2.2806 \times 10^{11}}\right] \begin{array}{lll} \times & 925\text{ mm} & = 9.130\text{ MPa (Top)} \\ \times & 904\text{ mm} & = 8.923\text{ MPa Bottom)} \end{array}$$

Barrier and future wearing surface: $$\left[\frac{(318+184)(1000)(1000)}{4.367 \times 10^{11}}\right] \begin{array}{lll} \times & 710\text{ mm} & = 0.816\text{ MPa (Top)} \\ \times & 1304\text{ mm} & = 1.499\text{ MPa (Bottom)} \end{array}$$

LL + IM: $$\left[\frac{(3528)(1000)(1000)}{4.367 \times 10^{11}}\right] \begin{array}{lll} \times & 710\text{ mm} & = 5.736\text{ MPa (Top)} \\ \times & 1304\text{ mm} & = 10.535\text{ MPa (Bottom)} \end{array}$$

Design Stresses (Top): $+7.422 + 8.615 + 0.259 + 5.736 = +22.032$ MPa

Design Stresses (Bottom): $-7.254 + (-8.419) + (-0.475) + (-10.535) = -26.683$ MPa

Design Stresses

Transfer: (Beam DL only)

$$\sigma_{t_{top}} = 7.42 \text{ MPa}$$

$$\sigma_{t_{bot}} = 7.25 \text{ MPa}$$

Final:

Compression Stresses:

$$\sigma_{f_{top}} = DC + DW = 7.42 + 9.13 + 0.82 = 17.37 \text{ MPa}$$
$$\sigma_{f_{top}} = 0.5(DC + DW) + LL + IM = 0.5(17.37) + 5.74 = 14.43 \text{ MPa}$$
$$\sigma_{f_{top}} = DC + DW + LL + IM = 17.37 + 5.74 = 23.11 \text{ MPa}$$

Tension Stresses:

$$\sigma_{f_{bot}} = DC + DW + 0.8(LL + IM) = -7.25 - 8.92 - 1.50 - 0.8(10.54) = -26.10 \text{ MPa}$$

Design Stress Table for Plans

BEAM DESIGN STRESSES		
Beam Length (C.L Brg. to C.L. Brg.) (Horizontal Distance)		35 000
Beam D.L. Stresses(MPa @ 0.5 pt)	F_t	7.42
	F_b	-7.25
Compression Zone – DC + DW (MPa)	F_t	17.37
Compression Zone – 0.5 (DC + DW) + LL +IM (MPa)	F_t	14.43
Compression Zone – DC + DW + LL + IM (MPa)	F_t	23.11
Tension Zone – DC + DW + 0.8 (LL + IM) (MPa)	F_b	-26.10
Factored Moment @ Section (kN-m)	$*M_u$	11 860

$*M_u = 1.0\,[1.25\,DC + 1.50\,DW + 1.75\,(LL + IM)]$

Stresses Due to Prestress

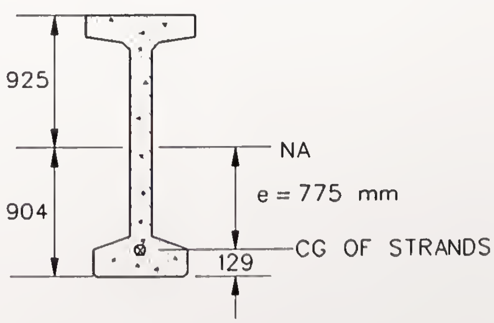
Try 48 strands:

$e = 775$ mm

$A_s = 98.77 \text{ mm}^2$

$f_{pu} = 1860$ MPa

Estimate losses at transfer to be 8%.



$$P_t = 48(98.77\ \text{mm}^2)\ (0.92)(0.75)(1860\ \text{MPa}) = 6\,084\,550\ \text{N}$$

$$\sigma_t = \frac{P}{A} \pm \frac{P_e y}{I}$$

$$\sigma_{t_{top}} = \frac{6\,084\,590\ \text{N}}{507\,000\ \text{mm}^2} - \frac{(6\,084\,590\ \text{N})\,(775\ \text{mm})\,(925\ \text{mm})}{2.2806\ \text{x}\ 10^{11}\ \text{mm}^4} = -\,7.1\ \text{MPa}$$

$$\sigma_{t_{bot}} = \frac{6\,084\,590\ \text{N}}{507\,000\ \text{mm}^2} + \frac{(6\,084\,590\ \text{N})\,(775\ \text{mm})\,(904\ \text{mm})}{2.2806\ \text{x}\ 10^{11}\ \text{mm}^4} = 30.7\ \text{MPa}$$

Use Approximate Lump-Sum Estimate at Time-Dependent Losses (LRFD Table 5.9.5.3-1):

$$230\left[1 - 0.15\left(\frac{f'_c - 41}{41}\right)\right] + 41\ \text{PPR}$$

$$= 230\left[1 - 0.15\left(\frac{48 - 41}{41}\right)\right] + 41\,(1) = 265$$

For low-relaxation strands, value may be reduced by 41 MPa:

$$P_f = 48\ (98.77\ \text{mm}^2)\ [(0.92)(0.75)(1860) - (265 - 41)] = 5\,022\,600\ \text{N}$$

$$\sigma_{f_{top}} = \frac{5\,022\,600\ \text{N}}{507\,000\ \text{mm}^2} - \frac{(5\,022\,600\ \text{N})\,(775\ \text{mm})\,(925\ \text{mm})}{2.2806\ \text{x}\ 10^{11}\ \text{mm}^4} = -\,5.9\ \text{MPa}$$

$$\sigma_{f_{bot}} = \frac{5\,022\,600\ \text{N}}{507\,000\ \text{mm}^2} + \frac{(5\,022\,600\ \text{N})\,(775\ \text{mm})\,(904\ \text{mm})}{2.2806\ \text{x}\ 10^{11}\ \text{mm}^4} = 25.3\ \text{MPa}$$

Check Stresses at Transfer

Compression Stresses (LRFD Article 5.9.4.1.1):

Service I Load Combination: $f_{allow} = 0.60\ f'_{ci} = 0.6\ (41.5) = 24.9$ MPa
$-7.3 + 30.7 = 23.4 \leq 24.9$ OK

Tension Stresses (LRFD 5.9.4.1.2):

Service III Load Combination: $f_{allow} = 0.25\ \sqrt{f'_{ci}} \leq 1.38$ MPa
$= 1.61$; therefore, 1.38 MPa controls

$7.4 + (-7.1) = 0.3 \geq -1.38$ MPa OK

Check Stresses at Final

Compression Stresses (LRFD Article 5.9.4.2.1):

Service I Load Combination: $f_{allow} = 0.45\ f'_c = 0.45\ (48) = 21.6$ MPa

$17.4 - 5.9 = 11.5 \leq 21.6$ OK

$f_{allow} = 0.40\ f'_c = 19.2$ MPa

$14.4 - \dfrac{5.9}{2} = 11.5 \leq 19.2$ OK

$f_{allow} = 0.60\ f'_c = 28.8$ MPa

$23.1 - 5.9 = 17.2 \leq 28.8$ OK

Tension Stresses (LRFD Article 5.9.4.2.2):

Service III Load Combination: $f_{allow} = 0.50\ \sqrt{f'_c} = 3.5$ MPa

$-26.1 + 25.3 = -0.8 \geq -3.5$ OK

Strength I Limit State:

$$\sum \eta_i \gamma_i Q_i = 1.25\ (1830 + 1910 + 55 + 318 + 215) + 1.5\ (183) + 1.75\ (3528) \quad \text{(LRFD Table 3.4.1-1)}$$
$$= 11\,859\ \text{kN} \bullet \text{m}$$

Nominal Flexural Resistance: (LRFD Article 5.7.3)

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \quad \text{(LRFD Equation 5.7.3.1.1-1)}$$

$$d_p = (1829 + 20 + 165) - 129$$
$$= 1885\ \text{mm}$$

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \quad \text{(LRFD Equation 5.7.3.1.1-2)}$$
$$= 2\left(1.04 - \frac{1675}{1860}\right)$$
$$= 0.279$$

$$\beta_1 = 0.85 - 0.5\left(\frac{31 - 28}{7}\right) = 0.83 \quad \text{(LRFD Article 5.7.2)}$$

Assume rectangular section behavior:

$$c = \frac{A_{ps}\, f_{pu}}{0.85\, f_c'\, \beta_1 b + K A_{ps} \dfrac{f_{pu}}{d_p}}$$ (LRFD Equation 5.7.3.1.1-4)

$$= \frac{(48)\,(98.77)\,(1860)}{0.85\,(31)\,(0.83)(2361) + (0.279)\,(48)\,(98.77)\,\dfrac{1860}{1885}}$$

= 166.6 mm (just greater than 165 mm; say ok)

$$f_{ps} = 1860\left(1 - 0.279\,\frac{166.6}{1885}\right)$$

= 1814 MPa

$a = \beta_1 c = (0.83)\ (166.6) = 138$ mm (LRFD Article 5.7.2.2)

$$M_n = A_{ps}\, f_{ps}\left(d_p - \frac{a}{2}\right)$$ (LRFD Equation 5.7.3.2.2-1)

$$= 48\,(98.77\ \text{mm}^2)\,(1814\ \text{MPa})\left[1885\ \text{mm} - \frac{138\ \text{mm}}{2}\right]$$

As is typical, the design is governed by the service limit states, not the strength limit states.

$$= 15\,618\ \text{kN} \bullet \text{m}\ (> 11\ 860\ \text{kN} \bullet \text{m})\ (\text{OK})$$

17.6 GIRDER DETAILS

17.6.1 Fabrication Lengths

The overall length of fabricated girders shall be increased 0.6 mm per m of length to compensate for elastic shortening, shrinkage and creep.

The length of fabricated girders shall also be adjusted for highway grades.

17.6.2 Camber and Dead-Load Deflection

Dead-load deflections shall be tabulated on erection plan drawings. A typical dead-load deflection table is shown in Figure 17.6A.

A note appears on Standard Drawings MSL-5 and MSL-6 directing the contractor's attention to the fact that the camber of the individual prestressed girders may vary. The contractor shall account for this variation in camber, determined in the field at 0.1 points, by varying the haunch depth as required.

17.6.3 Girder End Details

Typical girder end details, shear reinforcement and confinement steel are shown for the standard girders on the Standard Drawings MB-1, MB-A, MB-4, MM-72 and MMT-28. Modified girder end details will be required on the plans if the bridge is constructed on a skew.

17.6.4 Bearing Shoes

Fixed bearing shoes shown on Standard Drawings MB-1, MB-A, MB-4, MM-72 and MMT-28 shall be utilized respectively unless otherwise directed. If expansion shoes are required, they are designed specifically to the site conditions and appropriate details are shown on the plans.

17.6.5 Sole Plates

For highway grades greater than or equal to 2%, sole plates shall be beveled to allow for the typically level girder seats.

DEAD LOAD DEFLECTION TABLE					
TYPE 'A' PRESTRESSED CONCRETE BEAM (mm)					
Span Length	Tenth Point				
	0.1	0.2	0.3	0.4	0.5
16 000	4	7	9	11	12

Note: Deflections are symmetrical about the 0.5 point and do not include beam dead load.

TYPICAL DEAD-LOAD DEFLECTION TABLE

Figure 17.6A

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Chapter Eighteen
STRUCTURAL STEEL SUPERSTRUCTURES

Chapter Eighteen discusses structural steel provisions in Section 6 of the **LFRD Bridge Design Specifications** that require amplify-cation, clarification and/or an improved applica-tion. The Chapter is structured as follows:

1. Section 18.1 provides general information, mostly relating to cost-effective design practices, for which there is not a direct reference in Section 6 "Steel Structures" of the LRFD Specifications.

2. Sections 18.2 through 18.7 provide information which augments and clarifies Section 6 of the Specifications. To assist in using these Sections, references to the Specifications are provided, where applicable.

Chapter Thirteen of the **Montana Structures Manual** provides criteria for the general site considerations for which structural steel is appropriate. This includes span lengths, depth of superstructure, girder spacing, geometrics, seismic, aesthetics and cost. Chapter Eighteen addresses the detailed design of steel superstructures. Furthermore, the discussion in Chapter Eighteen is restricted to multi-girder steel superstructures. This reflects the popularity of these systems because of their straightforward design, ease of construction and aesthetically pleasing appearance. In addition, with some exceptions, the State of Montana lacks major waterways and/or large ravines that would require trusses, arches or suspension systems. Rigid frames have also been omitted because of their expensive fabrication.

18.1 GENERAL

18.1.1 Economical Steel Superstructure Design

Factors that influence the cost of a steel girder bridge include, but are not limited to, the number of girders, the type of material, type of substructure, amount of material, fabrication, transportation and erection. The cost of these factors changes periodically in addition to the cost relationship among them. Therefore, the guidelines used to determine the most economical type of steel girder on one bridge must be reviewed and modified as necessary for the next bridge.

18.1.1.1 Number of Girders

Generally, the fewest number of girders in the cross section as compatible with deck design requirements provides the most economical bridge.

MDT typically uses a minimum of four girders in a bridge cross section. The use of three girders may be considered on low-volume facilities where bridge closure during re-decking is not a problem and the superstructure is not susceptible to impact from overheight vehicles below.

18.1.1.2 Girder Spacing

The optimum girder spacing for a bridge will generally be determined by dividing the distance between the exterior girders into the least number of equal spaces that meet the structural requirements of the design. MDT uses girder spacings between 1.5 m and 4.5 m for most typical multi-girder steel bridges.

The optimum location of the exterior girder is controlled by these factors:

1. Because it is more cost effective to use the same girder design for interior and exterior girders to minimize fabrication costs, the exterior girder should be located to yield similar total moments (dead load plus live load, etc.) as the interior girders. This is typically achieved with an overhang width of approximately 35% to 40% of the girder spacing.

2. On bridges carrying barrier rails, the space required for deck drains may have an effect on the location of the exterior girder lines.

Note: *These are general rules of thumb, and some judgment should be exercised to allow for an even beam spacing or specific design requirements.*

18.1.1.3 Steel Weight Curves

AISC has prepared Steel Weight Curves based on data obtained over several years for cost-effective girder designs; see Figure 18.1A. The curves are based on the use of the Load Factor Design provisions of the **Standard Specifications for Highway Bridges** and should be considered as an approximation for superstructures designed with the LRFD Specifications when the modification indicated for HS25 loading is made. The Steel Weight Curves should only be used to provide a preliminary estimate of steel weight and a rough check on the economy of new designs.

18.1.1.4 Arrangements

Where pier locations are flexible, optimize the span arrangement. Steel design should not necessarily be associated with the use of longer spans. In selecting an optimum span arrangement, it is critical in all cases to consider the cost of the superstructure and substructure together as a total system.

A balanced span arrangement for continuous spans, with end spans approximately 0.8 of the length of interior spans, results in the largest possible negative moments at the piers, and smaller resulting positive moments and girder deflections. As a result, the optimum depth of the girder in all spans will be nearly the same resulting in a much more efficient design.

18.1.1.5 Rolled Beams versus Plate Girders

For shorter bridges, rolled beams will be more economical than welded plate girders. The major steel producers no longer routinely produce rolled WF sections over 1 m in depth. If rolled beams over 1-m deep must be used, check with the National Steel Bridge Alliance (NSBA) to determine their availability and cost. Regardless, allow the fabricator the option of replacing the rolled beam with an equivalent welded plate girder; i.e., with the same plate thicknesses as the flange and the web of the rolled beam.

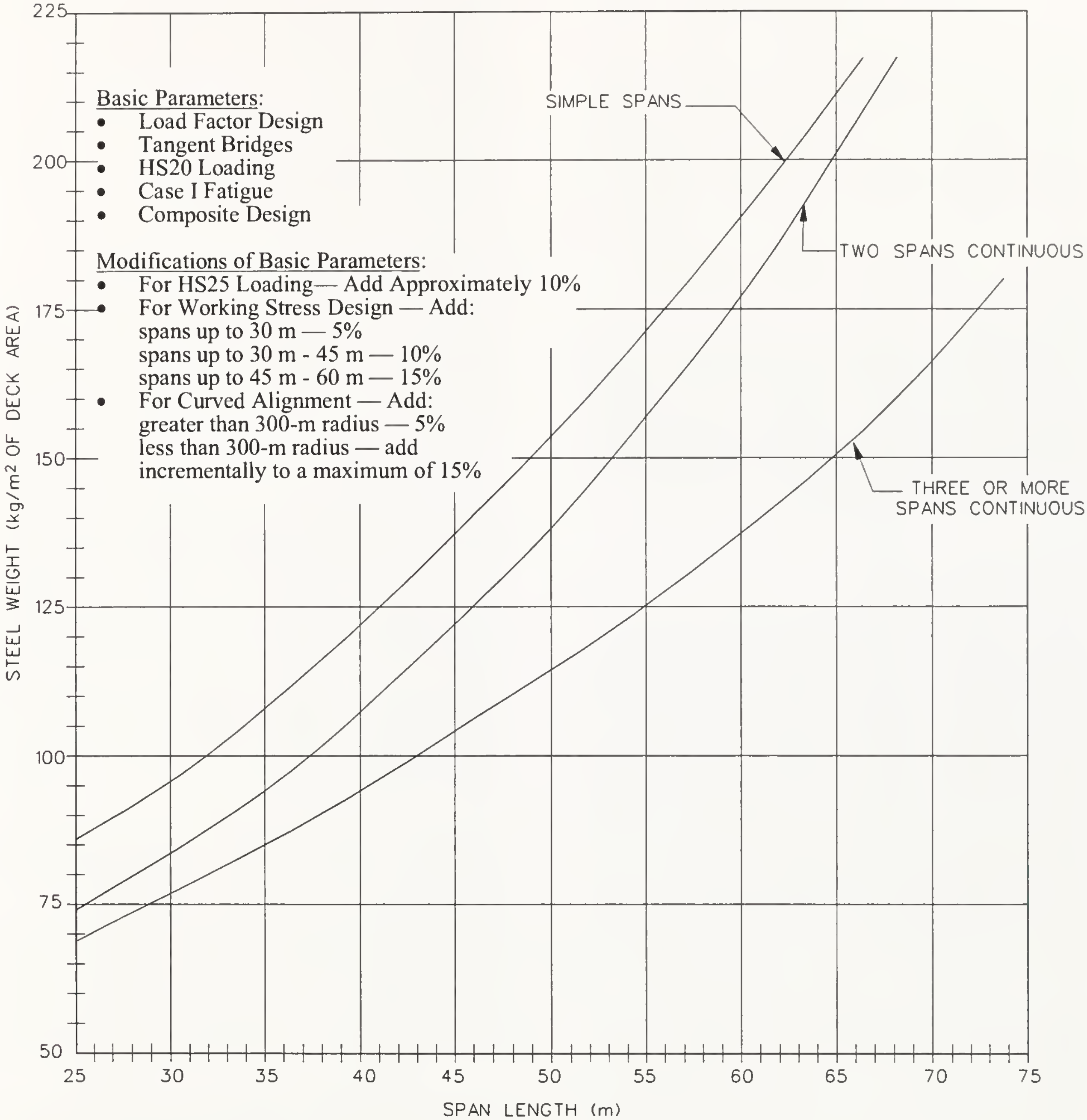
18.1.2 <u>Economical Plate Girder Design</u>

In addition to the information in the LRFD Specifications, the following applies to the design of structural steel plate girders.

18.1.2.1 General

Plate girders should be made composite with the bridge deck and continuous over interior supports where applicable.

To achieve economy in the fabrication shop, all girders in a multi-girder bridge should be identical with the critical girder, usually the interior girder, governing all girder designs. Identical girders, both interior and exterior, can be achieved by limiting overhang lengths as suggested in Section 18.1.1.2.



Notes: 1. *These curves apply to I-girders only.*

2. *For the curve labeled "Three or More Spans Continuous," a balanced span arrangement is assumed (end span equal to approximately 0.8 of the interior spans), and the interior span length should be used with the curve.*

STEEL WEIGHT CURVES

Figure 18.1A

18.1.2.2 Haunched Girders

When practical, constant-depth girders shall be used. Haunched girders are generally uneconomical for spans less than 120 m. They may be used where aesthetics or other special circumstances are involved, but constant-depth girders will generally be more cost-effective.

18.1.2.3 Longitudinally Stiffened Webs

Longitudinally stiffened webs are generally uneconomical for spans less than 90 m. The ends of longitudinal stiffeners are fatigue sensitive if subject to applied tensile stresses. Therefore, they must be ended in zones of little or no applied tensile stresses. In general, do not use longitudinally stiffened webs.

18.1.2.4 Field Splices

Field splices are used to reduce shipping lengths, but they are expensive and their number should be minimized. Field sections should not exceed 38 m in length and 82 000 kg in weight without investigation of permits and shipping constraints. As a general rule, the unsupported length in compression of the shipping piece divided by the minimum width of the flange in compression in that piece should be less than approximately 85. It is a good design practice to reduce the flange cross sectional area by no more than approximately 25% of the area of the heavier flange plate at field splices to reduce the build-up of stress at the transition.

18.1.2.5 Size of Flange

The minimum flange plate size for built-up girders is 300 mm x 20 mm. Figure 18.1B presents commonly specified metric plate thicknesses. Designers may use a flange width of approximately 20% to 25% of the web depth as a rule of thumb for an initial trial design. Designers should use as wide a flange girder plate as practical consistent with stress and b/t requirements. This contributes to girder stability and reduces the number of passes and weld volume at flange butt welds. Flange widths should always be an even number of mm to avoid 0.5-mm widths when working to flange centerlines; preferably, they should be in increments of 50 mm. The desirable maximum flange thickness is 80 mm.

Metric Units (mm)	Equivalent English Units (inches)	Metric Units (mm)	Equivalent English Units (inches)
5	0.1969	28	1.1024
6	0.2362	30	1.1811
7	0.2756	32	1.2598
8	0.3150	35	1.3780
9	0.3543	38	1.4961
10	0.3937	40	1.5748
11	0.4331	45	1.7717
12	0.4724	50	1.9685
14	0.5512	55	2.1654
16	0.6299	60	2.3622
18	0.7087	60 mm - 200 mm, use 10-mm increments.	
20	0.7874	> 200 mm, use 50-mm increments.	
22	0.8661	Based on ANSI Standard B32.3. Mills can produce	
25	0.9843	any metric plate size upon request.	

METRIC PLATE THICKNESSES

Figure 18.1B

18.1.2.6 Shop Splices

Use no more than two shop splices (three plate thicknesses) in the top or bottom flange within a single field section. The designer should maintain constant flange widths within a field section for economy of fabrication. In determining the points where changes in plate thickness occur within a field section, the designer should weigh the cost of butt-welded splices against extra plate area. In many cases, it may be advantageous to continue the thicker and/or wider plate beyond the theoretical step-down point to avoid the cost of the butt-welded splice. Locate shop splices at least 150 mm away from web splices or transverse stiffeners to facilitate testing of the weld.

An understanding of the most economical way of producing flanges in the shop makes this easier to understand. The most efficient way to construct flanges is to butt-weld together several wide plates of varying thicknesses received from the mill. After welding and non-destructive testing, the individual flanges are "stripped" from the full plate (Figure 18.1C). This reduces the number of welds, individual runoff tabs to both start and stop welds, the amount of material waste and the number of X-rays for non-destructive testing. The obvious objective, therefore, is for flange widths to remain constant within an individual shipping length by varying material thickness as required. Constant flange width within a field section may not always be practical in girder spans over 100 m where a flange width transition may be required in the negative bending regions.

Because structural steel plate is most economically purchased in widths of at least 1200 mm, it is advantageous to repeat plate thicknesses as much as practical. In the example shown in Figure 18.1C, many of the plates of like width could be grouped by thickness to meet the minimum 1200-mm width purchasing requirement, but the thicker plates do not allow this. In addition, all but the 75-mm plates shown are unique, thereby requiring additional material costs when purchasing plates. Furthermore, each splice must be individually, rather than gang, welded.

The discussion of flange design leads to the question of how much additional flange material can be justified to eliminate a width or thickness transition. Based on the experience of fabricators, some rules of thumb have been developed. Approximately 590 kg of steel should be saved to justify the cost of a transition in a flange plate.

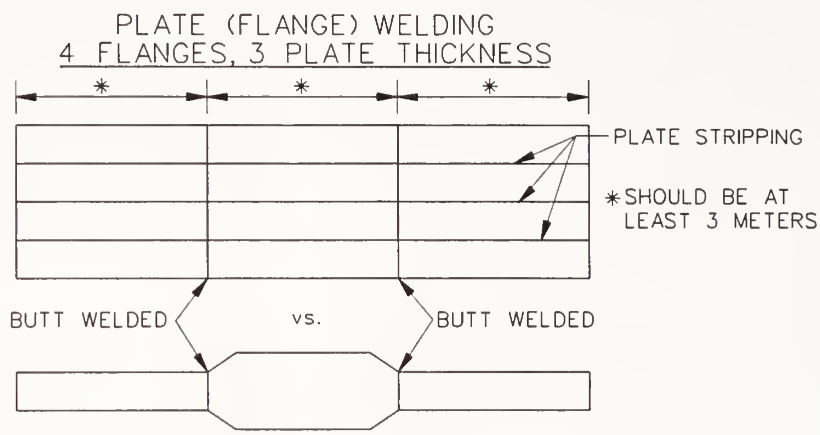
Though not preferred, if a transition in width must be provided, shift the butt splice a minimum of 75 mm from the transition as shown in Figure 18.1C. This makes it much easier to fit run-off tabs, weld and test the splice and then grind off the run-off tabs.

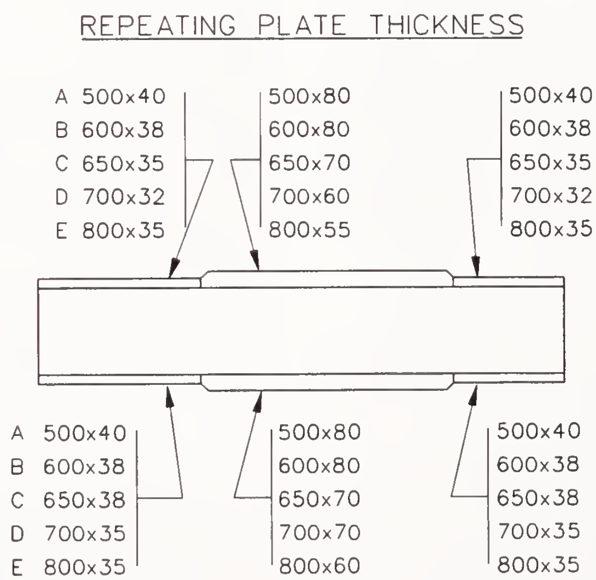
18.1.2.7 Web Plates

Where there are no depth restrictions, the web depth should be optimized. Designers may use preliminary design services available through the NSBA and Bethlehem Steel for the optimization of the web depth. Other programs or methods may also be used if they are based upon material use and fabrication unit costs. Web depths should always be an even number of mm, preferably in increments of 50 mm. The minimum web thickness should be 11 mm. Web thickness should not be changed by less than 5 mm.

Web design can have a significant impact on the overall cost of a plate girder. Considering material costs alone, it is desirable to make girder webs as thin as design considerations will permit. However, this will not always produce the greatest economy because fabricating and installing stiffeners is one of the most labor-intensive of shop operations. The following guidelines apply to the use of stiffeners:

1. Transversely unstiffened webs are generally more economical for web depths approximately 1250 mm or less.





DETAILS FOR FLANGE TRANSITIONS

Figure 18.1C

2. Between 1250-mm and 1800-mm depths, consider options for a partially stiffened and unstiffened web. A partially stiffened web is defined as one whose thickness is 1.5 mm less than allowed by specification for an unstiffened web at a given depth. Above 1800 mm, consider options for partially stiffened and fully stiffened webs.

18.1.3 Continuous Structures

Span-by-span continuity enhances both the strength and rigidity of the structure. One important benefit of structural continuity is the reduction in the number of deck expansion joints. Open or leaking deck joints may cause extensive damage to girder ends, diaphragms, bearings, bent caps and pier caps. See Chapter Fifteen for more discussion on bridge deck expansion joints.

18.1.4 Horizontally Curved Members

The design of horizontally curved structural steel beams shall be based on the AASHTO **Guide Specifications for Horizontally Curved Highway Bridges**. The following provides additional information.

18.1.4.1 General

The effect of curvature must be accounted for in the design of all steel superstructures where the components are fabricated on horizontal curves. The magnitude of the effect of horizontal curvature is primarily a function of the curve radius, girder spacing, span, diaphragm spacing and, to a lesser degree, depth and flange proportions. The effect of curvature develops in two ways that are either nonexistent or insignificant in tangent bridges. First, the general tendency is for each girder to overturn, which has the effect of transferring both dead and live load from one girder to another transversely. The net result of this load transfer is that some girders carry more load and others less. The load transfer is carried through the diaphragms and the deck. The second effect of curvature is the concept of flange bending caused by torsion in curved components being almost totally resisted by horizontal shear in the flanges. The horizontal shear results in moments in the flanges. The stresses caused by these moments either add to or reduce the stresses from vertical bending. The torsion also causes warping of the girder webs.

The LRFD Specifications currently do not include design provisions for horizontally curved steel bridges. Until such time as LRFD curved girder provisions are developed, any bridge containing a curve segment must be designed by load factor design using the current HS loadings.

18.1.4.2 Methods of Analysis

All curved systems should be analyzed for design by a rigorous structural method or by the V-load method that was published as Chapter 12 of the **USS Highway Structures Design Handbook**, except where curvature is within the limits as specified in Article 4.6.1.2.1 of the LRFD Specifications.

18.1.4.3 Diaphragms, Bearings and Field Splices

Cross frames and diaphragms shall be considered primary members. All curved steel simple-span and continuous-span bridges should have their diaphragms directed radially except end diaphragms, which should be placed parallel to the centerline of bearings.

Design all diaphragms, including their connections to the girders, to carry the total load to be transferred at each diaphragm location. Cross frames and diaphragms preferably shall be as close as practical to the full depth of the girders.

For ordinary geometric configurations, no extra consideration need be given to the unique expansion characteristics of curved structures. On occasion, in some urban metropolitan areas

(but rarely in Montana), wide sharply curved structures are required. In these circumstances, the designer must consider multi-rotational bearings and selectively providing restraint either radially and/or tangentially to accommodate the thermal movement of the structure.

Design the splices in flanges of curved girders to carry flange bending or lateral bending stresses and vertical bending stresses in the flanges.

Flange tip stresses shall be governed by the Load Factor Design Specifications for allowable stresses and fatigue allowables.

18.1.5 Integral End Bents

Chapter Nineteen discusses the design of integral end bents. The following applies to the use of integral end bents in combination with steel superstructures:

1. Bridge Length. Integral end bents empirically designed may be used with structural steel bridges where the movement does not exceed 50 mm at the abutment and the skew does not exceed 20°. Longer expansion lengths may be used if rational analysis of induced pile stresses indicates that the piles are not overstressed.

2. Deck Pour. Place an interior diaphragm within 3 m of the end support to provide beam stability during the deck pour.

3. Anchorage. Steel beams and girders should be anchored to the concrete cap. A minimum of three holes should be provided through the webs of steel beams or girders to allow #19 bars to be inserted to anchor the beam to the backwall.

18.1.6 Falsework

Steel superstructures should generally be designed with no intermediate falsework during placing and curing of the concrete deck slab.

18.2 MATERIALS

Reference: LRFD Article 6.4

18.2.1 Structural Steel

Reference: LRFD Article 6.4.1

18.2.1.1 Material Type

The most cost-effective choice of steel is unpainted M270 Grade 345W. Recently, HPS 485W steel has been developed that may prove to be as cost effective as 345W. Determine availability, fabrication and estimated cost comparisons to 345W and secure the Bridge Design Engineer's approval before proceeding with HPS 485W design. The initial cost advantage of 345W compared to painted, high-strength steel (e.g., M270 Grade 345) can range up to 15%. When future repainting costs are considered, the cost advantage is more substantial. This reflects, for example, environmental considerations in the removal of paint, which can make the use of painted steel cost prohibitive.

For long-span girder bridges, the more cost-effective solution may be M270 Grade HPS 485W. The premium on material costs is offset by a savings in tonnage. The most cost-effective HPS 485W design solutions tend to be hybrid girders with Grade 345 webs with 485W tension and compression flanges in the negative-moment regions and tension flanges only in the positive-moment regions.

Despite its cost advantage, the use of unpainted weathering steel is not appropriate in all environments and at all locations. The application of weathering steel and its potential problems are discussed in depth in FHWA Technical Advisory "Uncoated Weathering Steel in Structures," October 3, 1989. Also, the proceedings of the "Weathering Steel Forum," July 1989, are available from the FHWA Office of Implementation, HRT-10. Unpainted weathering steel should not be used where any of the following adverse conditions exist:

1. Environment. Unpainted weathering steel should not be used in industrial areas where concentrated chemical fumes may drift onto the structure. If in doubt, its suitability should be determined by a corrosion consultant from the steel industry.

2. Location. Unpainted weathering steel should not be used at grade separations in "tunnel" conditions, which are produced by depressed roadway sections with narrow shoulders between vertical retaining walls, with shallow vertical clearances and with deep abutments adjacent to the shoulders. This "tunnel" effect prevents roadway spray from being dissipated and spread by air currents. Note that there is no evidence of salt spray corrosion where the longitudinal extent of the vertical walls is limited to the abutment itself, and roadway spray can be dissipated on both approaches.

3. Water Crossings. Sufficient clearance over bodies of water should be maintained so that water vapor condensation does not result in prolonged periods of wetness on the steel. In Montana, these situations may occur over stagnant overflow channels or irrigation canals that run full during the irrigation season. If freeboard is minimal and the bridge is wide, do not use weathering steel unless there are geometric limitations that preclude the use of other sections.

Where unpainted weathering steel is inappropriate, and a concrete alternative is not feasible, the most economical painted steel is AASHTO M270 Grade 345 steel in both webs and flanges. These are less expensive than Grade 250 designs. Hybrid designs, such as Grade 345 in flanges and Grade 250 in webs, seldom result in significant economy. The theoretical economy is not achievable because the nominal shear resistance of homogeneous sections is computed by summing the contributions of beam action and the post-buckling, tension-field action. Tension-field action is not currently permitted for hybrid sections.

The FHWA Technical Advisory "Uncoated Weathering Steel in Structures" is an excellent source of information, but its recommendation for partial painting of the steel in the vicinity of deck joints should not be considered the first choice. The best solution is to eliminate deck joints. If a joint is used, consideration should be given to painting all superstructure steel within 3 m of the joint. In shorter bridges, the end joint should be replaced by an integral end bent (see Chapter Nineteen).

18.2.1.2 Details for Unpainted Weathering Steel

When using unpainted weathering steel, the following drainage treatments should be considered to avoid premature deterioration:

1. A groove should be provided at the end of the deck overhang.

2. The number of bridge deck drains should be minimized, the drainage pipes should be generous in size, and they should extend below the steel bottom flange.

3. Eliminate details that serve as water and debris "traps." Seal or paint overlapping surfaces exposed to water. This applies to non-slip-critical bolted joints. Slip-critical bolted joints or splices should not produce "rust-pack" when the bolts are spaced according to the LRFD Specifications and, therefore, do not require special protection.

4. Consider protecting pier caps and abutment walls with a concrete sealer to minimize staining.

5. Consider wrapping the piers and abutments during construction to minimize staining while the steel is exposed to rainfall.

6. Place a bead of caulk or other material transverse across the top of the bottom flange in front of the substructure elements to prevent water from running off of the flange onto the concrete.

18.2.1.3 Charpy V-Notch Fracture Toughness

Reference: LRFD Article 6.6.2

The temperature zone appropriate for using LRFD Table 6.6.2-1 for the State of Montana is Temperature Zone 3.

18.2.2 Bolts

Reference: LRFD Article 6.4.3

For normal construction, high-strength bolts shall be:

1. Unpainted Weathering Steel: 22 mm A325 M (Type 3); Open Holes: 25 mm

2. Painted Steel: 22 mm A325M (Type 1); Open Holes: 25 mm

18.2.3 Other Structural Elements

Reference: None

Grade 250 steel shall not be used for secondary members when unpainted weathering steel is used in the web and flanges. In all cases, steel for all splices shall be the same material as used in the web and flanges of built-up girders.

For steel bridges that will be painted, the designer must specify in a special provision which color will be used. Even when using unpainted weathering steel, a color must be specified if any of the steel will be painted.

18.3 LOADS

18.3.1 Limit States

See Section 14.1.3 for a discussion on the load side of the basic LRFD equation that represents all limit states.

18.3.2 Distribution of Dead Load

See Section 14.2.4 for a discussion on the distribution of dead load.

18.3.3 Live-Load Deflection

Reference: LRFD Articles 2.5.2.6.2 and 3.6.1.3.2

Limitations on live-load deflections are optional in the LRFD Specifications. However, MDT has elected to apply the traditional limitation on live load plus dynamic load allowance deflections of 1/800 of the span length for the design of steel rolled beam and plate girder structures of simple or continuous spans. For structures in urban areas used by pedestrians and/or bicyclists, live-load deflections should be limited to 1/1000 of the span length. The span lengths used to determine deflections shall be assumed to be the distance between centers of bearings or other points of support.

Live-load deflections should be evaluated using the provisions of Articles 2.5.2.6.2 and 3.6.1.3.2 in the LRFD Specifications. In calculating live load deflections, start with a composite section that uses the gross cross-sectional area of the deck. Assume that section applies for the length of the girder. Divide the width of the concrete section by the ratio of the elastic moduli (E_s/E_c) to transform the section before calculating deflections. In effect, the distribution of live loads is the number of loaded lanes divided by the number of girders. The concrete deck should be considered to act compositely with the girder even though sections of the girder may not be designed as composite.

For horizontally curved girders, uniform participation of the girders should not be assumed. Instead, the live load should be placed to produce the maximum deflection in each girder individually in the span under consideration. When multiple lanes are loaded, multiple presence factors should be applied.

18.4 FATIGUE CONSIDERATIONS

Reference: LRFD Article 6.6

In Article 6.6.1, the LRFD Specifications categorizes fatigue as either "load induced" or "distortion induced." Actually, both are load induced, but the former is a "direct" cause of loading, and the latter is an "indirect" cause in which the force effect, normally transmitted by a secondary member, may tend to change the shape of, or distort, the cross section of a primary member.

18.4.1 Load-Induced Fatigue

Reference: LRFD Article 6.6.1.2

Article 6.6.1.2 provides the framework to evaluate load-induced fatigue. Section 18.4.1 provides additional information on the implementation of Article 6.6.1.2 and defines MDT's interpretation of the LRFD provisions.

Load-induced fatigue is determined by the following:

1. the stress range induced by the specified fatigue loading at the detail under consideration;

2. the number of repetitions of fatigue loading a steel component will experience during its 75-year design life. This is determined by anticipated truck volumes; and

3. the nominal fatigue resistance for the Detail Category being investigated.

18.4.1.1 Fatigue Stress Range

The following applies:

1. Range. The fatigue stress range is the difference between maximum and minimum stresses at a detail subject to a net tensile stress caused by a single design truck which can be placed anywhere on the deck within the boundaries of a design lane. If a refined analysis method is used, the design truck shall be positioned to maximize the stress in the detail under consideration. The design truck should have a constant 9-m spacing between the 145-kN axles. The dynamic load allowance is 0.15, and the fatigue load factor is 0.75.

2. Regions. Fatigue should only be considered in those regions of a steel member either having a net applied tensile stress, or where the unfactored permanent loads produce a compressive stress less than twice the maximum fatigue tensile stress.

3. Analysis. Unless a refined analysis method is used, the single design lane load distribution factor in LRFD Article 4.6.2.2 should be used to determine fatigue stresses. This tabularized distribution-factor equations incorporate a multiple presence factor of 1.2, which should be removed by dividing either the distribution factor or the resulting fatigue stresses by 1.2. This division does not apply to distribution factors determined using the lever rule.

18.4.1.2 Stress Cycles

Article 6.6.1.2.5 of the LRFD Specifications defines the number of stress cycles (N) as:

$$N = [(365)(75)(ADTT)(p)]\,(n) \qquad \text{(Eq. 18.4.1)}$$

Where:

ADTT = Average Daily Truck Traffic = the number of trucks per day headed in one direction "averaged" over the design life of the structure. The Department's method of "averaging" is described in the following example problems.

p = the maximum fraction of the total ADTT which will occupy a single lane. As defined in Article

3.6.1.4.2, if one direction of traffic is restricted to:

- 1 lane $p = 1.00$
- 2 lanes $p = 0.85$
- 3 or more lanes $p = 0.80$

n = number of stress range cycles per truck passage. As defined in Article 6.6.1.2.5, for simple and continuous spans not exceeding 12 m, n = 2.0. For spans greater than 12 m, n = 1.0, except at locations within 0.1 of the span length from a continuous support, where n = 1.5.

The term in the brackets of Equation 18.4.1 represents the total accumulated number of truck passages in a single lane during the 75-year design life of the structure. Traffic volumes will, of course, increase over time. See the Project File for traffic growth numbers for a given project. If there is any doubt on the annual growth rates, contact the Rail, Transit and Planning Division.

Examples 18.4.1 and 18.4.2 illustrate the application of traffic growth rates to determine the total fatigue live-load cycles over the 75-year design life of the structure.

18.4.1.3 Fatigue Resistance

Article 6.6.1.2.3 of the LRFD Specifications groups the fatigue resistance of various structural details into eight categories (A through E′). Experience indicates that Detail Categories A, B and B′ are seldom critical. Investigation of details with a fatigue resistance greater than Category C is appropriate only in unusual design cases. For example, Category B applies to base metal adjacent to slip-critical bolted connections and should only be evaluated when thin splice plates or connection plates are used. The Specifications requires that the fatigue stress range for Detail Categories C through E′ must be less than the fatigue resistance for each respective Category.

The fatigue resistance of a category is determined from the interaction of a Category Constant "A" and the total number of stress cycles "N" experienced during a 75-year design life of the structure. This resistance is defined as $(A/N)^{1/3}$. A Constant Amplitude Fatigue Threshold is also established for each Category. If the applied fatigue stress range is less than ½ of the threshold value, the detail has infinite fatigue life.

Fatigue resistance is independent of the steel strength. The application of higher grade steels causes the fatigue stress range to increase, but the fatigue resistance remains the same. These imply that fatigue may become a more controlling factor where higher strength steels are used.

* * * * * * * * *

<u>**Example 18.4.1**</u>

<u>Given:</u> 2-lane rural arterial
Current AADT = 3000 vpd (1500 vpd each direction)
Annual traffic growth rate = 1.5%
Percent trucks = 13%
Two spans, 50-m each
Connection plate located 10 m from the interior support
Unfactored DL stress in bottom flange = 28 MPa compression
Unfactored fatigue stresses in bottom flange using unmodified single-lane distribution factor = 27 MPa tension and 34 MPa compression

<u>Find:</u> Determine the fatigue adequacy at the toe of a transverse connection plate to bottom flange weld.

<u>Solution:</u>

Step 1: The LRFD Specifications classifies this connection as Detail Category C′. Therefore:

- Constant A = 14.4×10^{11} MPa^3 (LRFD Table 6.6.1.2.5-1)

- $(\Delta F)_{TH}$ = Constant Amplitude Fatigue Threshold = 82.7 MPa (LRFD Table 6.6.1.2.5-3)

Step 2: Compute the factored live-load fatigue stresses by applying dynamic load allowance and fatigue load factor, and removing the multiple presence factor:

- Tension: 27(1.15)(0.75)/1.2 =19.4 MPa
- Compression: 34(1.15)(0.75)/1.2 =24.4 MPa
  Fatigue Stress Range =43.8 MPa

Step 3: Determine if fatigue must be evaluated at this location:

- Net tension = (DL stress) – (Fatigue stress)
- Net tension = 28 MPa (Compressive) – 19.4 MPa (Tensile) = 8.6 MPa (Compressive)

Although there is no net tension at the flange, the unfactored compressive DL stress (28 MPa) does not exceed twice the tensile fatigue stress (38.8 MPa). Therefore, fatigue must be considered.

Step 4: Check for infinite life:

First, check the infinite life term. This will frequently control the fatigue resistance when traffic volumes are large. $(\Delta F)_n = ½(\Delta F)_{TH}$ = 0.5(82.7) = 41.35 MPa. Because the fatigue stress range (43.8 MPa) exceeds the infinite life resistance (41.35 MPa), the detail does not have infinite fatigue life.

Step 5: Compute the total number of truck passages over the design life of the structure for both directions of travel:

$$V_T = \frac{365\ V_o\ ((1+r)^t - 1)}{r} \qquad \text{(Eq. 18.4.2)}$$

Where:

V_T = Total number of truck passages (both directions)

V_o = Current average daily number of trucks (both directions)

r = Annual traffic growth rate, decimal

t = Design life of structure, years

For this Example:

V_o = (3000) (0.13) = 390
r = 1.5% = 0.015
t = 75 years

Equation 18.4.2 becomes:

$$V_T = \frac{(365)(390)((1+0.015)^{75} - 1)}{0.015}$$

$$V_T = 19.50 \times 10^6$$

Step 6: Compute the total number of truck passages per direction during the 75-year design life:

$$V_T / \text{Direction} = (19.50 \times 10^6)(0.5) = 9.75 \times 10^6$$

Step 7: Determine "p" and "n" for Equation 18.4.1:

- Because this is a two-lane structure carrying bi-directional traffic, all truck traffic headed in one direction can occupy only one lane. Therefore, p = 1.0. See LRFD Article 3.6.1.4.2.

- The span exceeds 12 m and the point being considered is located more than 0.1 of the span length away from the interior support. Therefore, n = 1.0. See Equation 18.4.1.

Step 8: Using Equation 18.4.1, compute the number of stress cycles:

$N = [(9.75 \times 10^6)(p)]\ (n)$
$N = [(9.75 \times 10^6)(1.0)]\ (1.0)$
$N = 9.75 \times 10^6$

Step 9: Compute the nominal fatigue resistance:

Fatigue Resistance		75-Year Life	Infinite Life
$(\Delta F)_n$	=	$(A/N)^{1/3}$	$½(\Delta F)_{TH}$

Step 10: Check to see if the detail will have at least a 75-year fatigue life:

$$(\Delta F)_n = (A/N)^{1/3}$$
$$= [(14.4 \times 10^{11})/(9.75 \times 10^{6})]^{1/3}$$
$$= 52.86 \text{ MPa}$$

The 75-year fatigue resistance (52.86 MPa) exceeds the fatigue stress range (43.8 MPa); therefore, the detail is satisfactory.

<u>Example 18.4.2</u>

<u>Given</u>: 2-lane freeway bridge carrying westbound traffic only
Current AADT = 15,000 vpd (7,500 vpd in WB lanes)
Percent trucks = 22%
Two-span continuous bridge, 50-m each
Area investigated is located 4 m from interior support
Unfactored DL stress in the top flange = 55 MPa Tension
Unfactored fatigue stresses in the top flange using unmodified single lane distribution factor = 39 MPa Tension and 6 MPa Compression

<u>Find</u>: Determine the fatigue adequacy of the top flange with welded stud shear connectors in the negative moment region.

<u>Solution</u>:

Step 1: The LRFD Specifications classifies this connection as Detail Category C. Therefore:

- Constant A = 14.4×10^{11} MPa^3 (LRFD Table 6.6.1.2.5-1)
- $(\Delta F)_{TH}$ = Constant Amplitude Fatigue Threshold = 69.0 MPa (LRFD Table 6.6.1.2.5-3)

Step 2: Compute the factored live-load fatigue stresses by applying dynamic load allowance and fatigue load factor, and removing the multiple presence factor:

- Tension: 39(1.15)(0.75)/1.2 = 28.0 MPa
- Compression: 6(1.15)(0.75)/1.2 = <u>4.3 MPa</u>

Fatigue Stress Range = 32.3 MPa

Step 3: Check for infinite life:

First, check the infinite life term (see Commentary C6.6.1.2.5 of the LRFD Specifications for a table of single-lane ADTT values for each detail category above which the infinite life check governs). This will frequently control the fatigue resistance when traffic volumes are large. $(\Delta F)_n = \frac{1}{2}(\Delta F)_{TH} = 0.5(69.0) = 34.50$ MPa. Because the fatigue stress range (32.3 MPa) is less than the infinite life resistance (34.50 MPa), the detail has infinite fatigue life and there is no need to check the 75-year fatigue life. The detail is satisfactory.

Provisions for investigating the fatigue resistance of shear connectors are provided in LRFD Articles 6.10.7.4.2 and 6.10.7.4.3.

* * * * * * * * * *

18.4.2 <u>Distortion-Induced Fatigue</u>

Reference: LRFD Article 6.6.1.3

Article 6.6.1.3 of the LRFD Specifications provides specific detailing practices for transverse and lateral connection plates intended to reduce significant secondary stresses which could induce fatigue crack growth. The provisions of the Specifications are concise and direct and require no mathematical computation; therefore, no further elaboration on distortion-induced fatigue is necessary.

18.4.3 <u>Other Fatigue Considerations</u>

Reference: Various LRFD Articles

The designer is responsible for ensuring compliance with fatigue requirements for all

structural details (e.g., stiffeners, connection plates, lateral bracing) shown on the plans.

In addition to the considerations in Section 18.4.1, the designer should be aware of the fatigue provisions in other Articles of Chapter 6 of the LRFD Specifications. They include:

1. Fatigue due to out-of-plane flexing in webs of plate girders — LRFD Article 6.10.6.

2. Fatigue at shear connectors — LRFD Articles 6.10.7.4.2 and 6.10.7.4.3.

3. Bolts subject to axial-tensile fatigue — LRFD Article 6.13.2.10.3.

18.5 GENERAL DIMENSION AND DETAIL REQUIREMENTS

Reference: LRFD Article 6.7

18.5.1 Design Information Table

For continuous structures, the drawings shall include a Design Information Table.

18.5.2 Dead-Load Camber

Reference: LRFD Article 6.7.2

18.5.2.1 General

Plate girders must be cambered to compensate for the vertical curve offset combined with the sum of load deflections due to composite and non-composite dead loads. Camber will be calculated to the nearest 1 mm. Dead load should include the weight of the steel, the deck and railing, and the future wearing surface. The effects of vertical curvature and superelevation should be considered. All beams and girders should be assumed to equally contribute to flexural resistance. Unfactored force effects should be used to determine the deflections. Calculated deflections are increased by 10% before entry into the Design Information Table to account for shrinkage and creep in the concrete.

18.5.2.2 Diagram

When the maximum total camber exceeds 5 mm, the plans must include a diagram and a table showing camber coordinates resulting from the effects listed in Section 18.5.2.1. Figure 18.5A provides an example of a camber table and an explanatory diagram.

The diagram shows vertical deflections with respect to a straight line ("string line") connecting the top of the girder web at the centerline of bearing at abutments and the top of the web at the centerline of bearing at intermediate bents. Bridges with variable superelevation must also have a String Line Slope Table that shows the slope of the string line for each span of each girder.

The camber table provides ordinates from the string line in millimeters at span tenth points and at girder field splices. At each of these locations, it provides an ordinate for the deflection due to the dead load of the girder and diaphragms alone and for the deflection due to total dead load. The table also provides an ordinate to match the roadway vertical curvature and the variation in the deck cross slope, then totals these ordinates at each location. The fabricator uses the total camber to shape the girder web. The contractor uses the two dead load ordinates to set deck forms.

Figure 18.5B shows the calculation of girder throw at the top of the web and at the top of the slab. These quantities provide the perpendicular offset between a vertical line and a line perpendicular to a tangent to the roadway surface. The angle separating the two lines matches the slope of the roadway vertical curvature at that location. The two lines intersect at the bottom of the web, called out as the Working Point in the Figure. The fabricator places bearing stiffeners along the line perpendicular to the profile grade tangent. Under load, the stiffeners will rotate to a vertical or more nearly vertical position. The plans show these throw quantities at each bearing in a table. Figure 18.5C contains an example of how this information appears on the plans.

18.5.3 Minimum Thickness of Steel

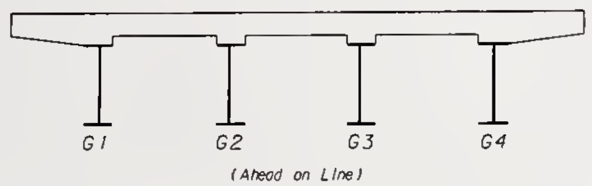
Reference: LRFD Article 6.7.3

The thickness of steel elements should not be less than:

1. Webs Cut From Plates: 11 mm.

2. Plate Girder Flanges: 20 mm.

TABLE OF DESIGN INFORMATION



O. L. = Dead Load
L. L. = Live Load
I. = Impact
T = Truck Loading
L = Lane Loading

(+) = Compression In Top Flange
(-) = Compression In Bottom Flange
Shears & Reactions In Kilonewtons
Moments In Kilonewton-Meters
Oeflections In Millimeters

		SPAN 1 AND SPAN 4		SPAN 2 AND SPAN 3		BENT 1 AND BENT 5		BENT 2 AND BENT 4		BENT 3	
		INTERIOR GIRDER	EXTERIOR GIRDER	INTERIOR GIRDER	EXTERIOR GIRDER	INTERIOR GIRDER	EXTERIOR GIRDER	INTERIOR GIRDER	EXTERIOR GIRDER	INTERIOR GIRDER	EXTERIOR GIRDER
Moment	O. L.	3135	2591	2162	1796	—	—	-7423	-6153	-7087	-5899
	L. L. + I.	3143 (T)	2494 (T)	3257 (L)	2581 (L)	—	—	-4024 (L)	-3218 (L)	-4355 (L)	-3483 (L)
Reaction or Shear	O. L.	779	647	748	624	416	344	1527	1271	1471	1229
	L. L. + I.	473 (L)	376 (L)	488 (L)	387 (L)	378 (L)	300 (L)	816 (L)	649 (L)	834 (L)	663 (L)
Oeflection	O. L.	73	60	62	51	—	—	—	—	—	—
	L. L. + I.	36	30	51	43	—	—	—	—	—	—

NOTES

Fabricate intermediate diaphragm stiffeners and web splices perpendicular to top girder flange.

D. L. deflections are directly proportional to D. L. weights which are as follows:
Total D. L. = 23.5 kN/m of Exterior Girder (Average)
Total D. L. = 28.2 kN/m of Interior Girder (Average)
D. L. Structural Steel = 5.99 kN/m of Girder (Average)

Total dead load deflections have been increased by 10% to allow for deflections due to shrinkage and creep of the concrete slab.

Deflections in the table preceded by a minus sign indicate upward deflections.

TABLE OF CAMBER INFORMATION

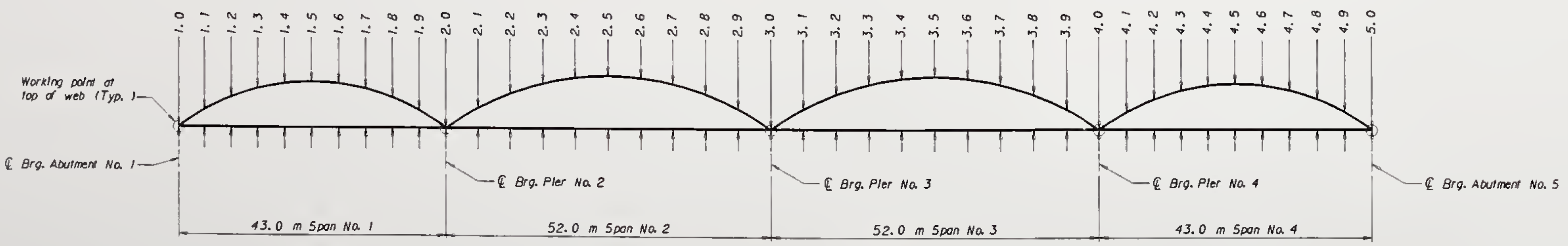
Location		SPAN NO. 1									
		1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9
Interior Girder	Diaphragm and Girder Deflection	0	5	10	13	14	13	10	6	3	2
	Total O. L. Deflection	0	29	52	68	73	68	56	38	19	6
Exterior Girder	Diaphragm and Girder Deflection	0	5	10	11	13	11	10	6	3	2
	Total O. L. Deflection	0	24	44	59	64	59	48	33	17	5

Location		SPAN NO. 2									
		2.0	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9
Interior Girder	Diaphragm and Girder Deflection	0	2	5	8	11	11	11	8	5	2
	Total O. L. Deflection	0	6	21	38	52	59	54	40	22	6
Exterior Girder	Diaphragm and Girder Deflection	0	2	5	8	10	11	10	8	5	2
	Total O. L. Deflection	0	5	17	33	46	51	46	35	19	6

Location		SPAN NO. 3									
		3.0	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.8	3.9
Interior Girder	Diaphragm and Girder Deflection	0	2	5	8	11	11	11	8	5	2
	Total O. L. Deflection	0	6	22	40	54	59	52	38	21	6
Exterior Girder	Diaphragm and Girder Deflection	0	2	5	8	10	11	10	8	5	2
	Total O. L. Deflection	0	6	19	35	46	51	46	33	17	5

Location		SPAN NO. 4									
		4.0	4.1	4.2	4.3	4.4	4.5	4.6	4.7	4.8	4.9
Interior Girder	Diaphragm and Girder Deflection	0	2	3	6	10	13	14	13	10	5
	Total O. L. Deflection	0	6	19	38	56	68	73	68	52	29
Exterior Girder	Diaphragm and Girder Deflection	0	2	3	6	10	11	13	11	10	5
	Total O. L. Deflection	0	5	17	33	48	59	64	59	44	24

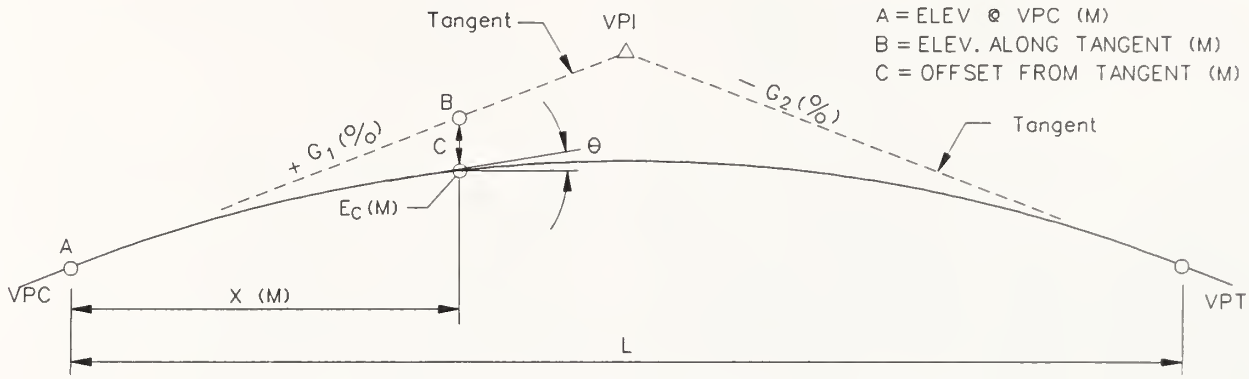
FIGURE 18.5A



GIRDER CAMBER DIAGRAM
No Scale

BRIDGE OVER MIDDLE FORK FLATHEAD RIVER
AT STA. 433+45.0
FEDERAL AID PROJECT NO. BR 1-2(86)180
FLATHEAD COUNTY

CAMBER DETAILS, DESIGN INFORMATION AND NOTES
No Scale

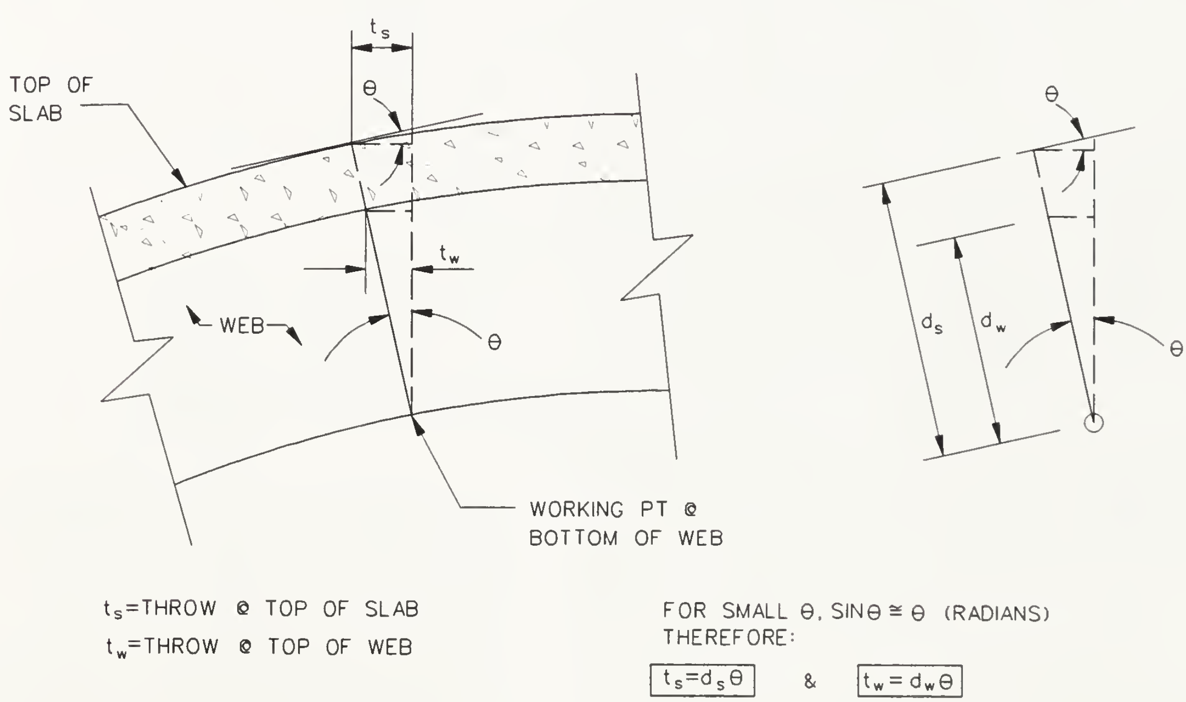


Curve Equation: $E_c = A + B + C$

$$E_c = E_{VPC} + G_1 \frac{X}{100} + \left(\frac{G_2 - G_1}{200L}\right) X^2$$

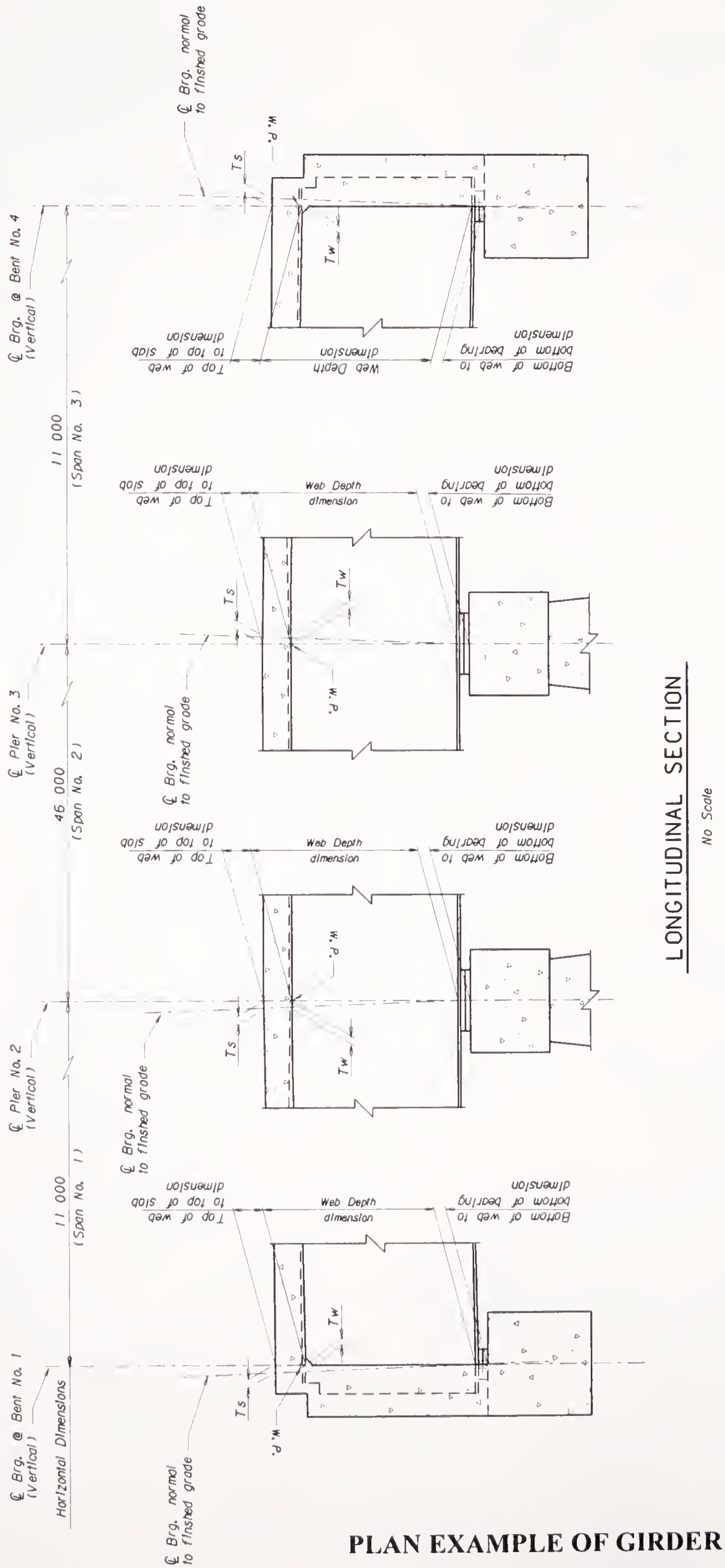
Slope of Curve @ Point X (Radians): $\theta = \dfrac{dE_c}{dx} = \dfrac{G_1}{100} + \dfrac{G_2 - G_1}{100L} X$

where: E_c = Elevation of Curve at Point X (m)



THROW CALCULATIONS

Figure 18.5B



GIRDER THROW CALCULATIONS

	Bent No. 1		Bent No. 2		Bent No. 3		Bent No. 4	
	Ts	Tw	Ts	Tw	Ts	Tw	Ts	Tw
Girder 1	21	18	0	0	0	0	17	15
Girder 2	14	12	0	0	0	0	13	11
Girder 3	7	6	0	0	0	0	7	6
Girder 4	1	1	0	0	0	0	0	0

Ts = Throw @ top of slab

Tw = Throw @ top of web

W. P. = Working Point

STRINGLINE SLOPE TABLE (%)

	Span No. 1	Span No. 2	Span No. 3
Girder 1	1.13	0	-0.89
Girder 2	0.74	0	-0.67
Girder 3	0.36	0	-0.35
Girder 4	0.04	0	-0.01

PLAN EXAMPLE OF GIRDER THROW

Figure 18.5C

18.5.4 Diaphragms and Cross-Frames

Reference: LRFD Articles 6.7.4 and 6.6.1.3.1

Diaphragms and cross-frames are vitally important in steel girder superstructures. They stabilize the girders during and after construction and distribute gravitational, centrifugal and wind loads. The spacing of diaphragms and cross-frames should be determined based upon the provisions of LRFD Article 6.7.4.1. As with most aspects of steel-girder design, the design of the spacing of diaphragms and cross-frames is iterative. A good starting point is the traditional maximum diaphragm and cross-frame spacing of 7.6 m. Most economical, modern steel girder designs will use spacings typically greater than 7.6 m.

18.5.4.1 General

The following applies to diaphragms and cross-frames:

1. Location. Place diaphragms or cross-frames at each support and throughout the span at an appropriate spacing. The location of the field splices should be planned to avoid any conflict between the connection plates of the diaphragms or cross-frames and the splice material.

2. Skew. Regardless of the angle of skew, place all intermediate diaphragms and cross-frames perpendicular to the girders. The intermediate diaphragms and cross-frames should be continuous across the cross section and not staggered.

3. End Diaphragms and Cross Frames. End diaphragms and cross-frames should be placed along the centerline of bearing. Set the top of the diaphragm below the top of the beam or girder to accommodate the joint detail and the thickened slab at the end of the superstructure deck, where applicable. The end diaphragms should be designed to support the edge of the slab including live load plus impact.

4. Curved-Girder Structures. Diaphragms or cross-frames connecting curved girders are considered primary members and should be oriented radially.

18.5.4.2 Diaphragm Details

On spans composed of rolled beams, diaphragms at continuous supports and at intermediate span points may be detailed as illustrated in Figure 18.5D. Figure 18.5E illustrates the typical end diaphragm connection details for rolled beams. Plate girders with web depths of 1060 mm or less should have the same diaphragm details. For plate girder webs more than 1060 mm deep, use cross-frames as detailed on Figure 18.5F.

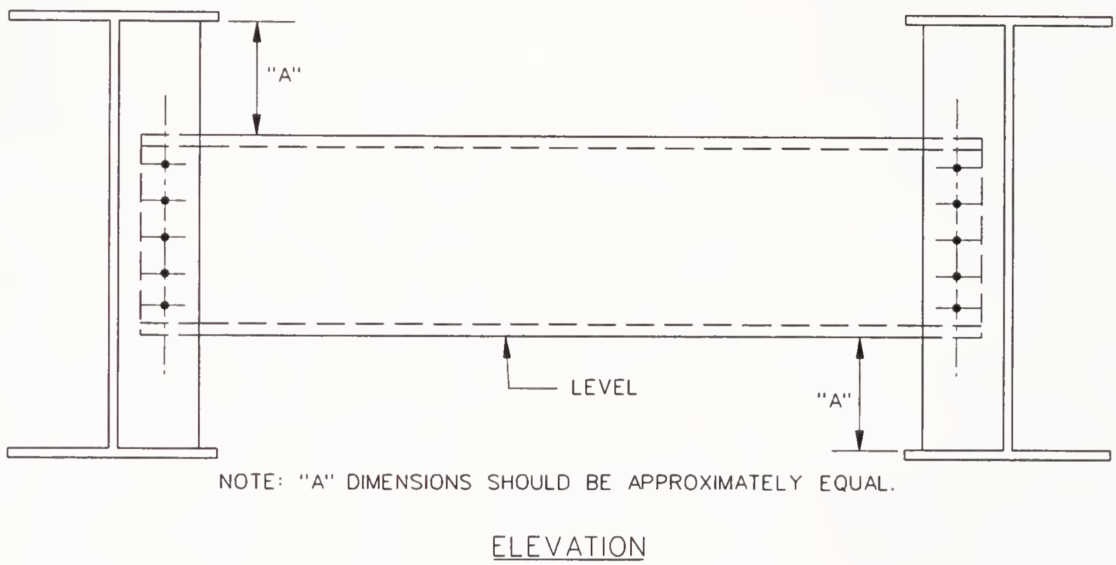
Intermediate diaphragms should be designed and detailed as nonload bearing. Diaphragms at points of support should be designed as a jacking frame, if needed, to support dead load only. See Section 18.5.5 to determine if the need exists. Jacking diaphragms should be considered at all supports when it is impractical to jack the girders directly.

18.5.4.3 Cross-Frame Details

Figure 18.5F illustrates typical cross-frame details. In general, the X-frame at the top of the Figure is more cost effective than the K-frame at the bottom. However, the K-frame should be used instead of the X-frame when the girder spacing becomes much greater than the girder depth and the “X” becomes too shallow.

Montana practice requires that cross-frame transverse connection plates, where employed, be welded to both the tension and compression flanges. The connection plate welds to the flanges should be designed to transfer the cross-frame forces into the flanges.

Connection plates should be fillet welded near side and far side to flanges. The flange welds should conform to the details shown in Figure 18.5G.

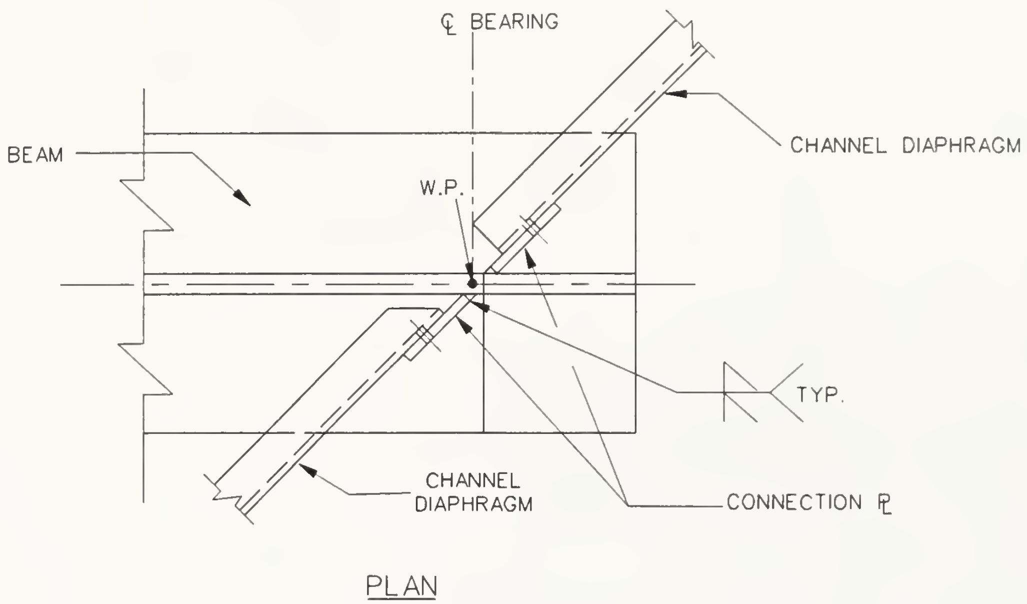


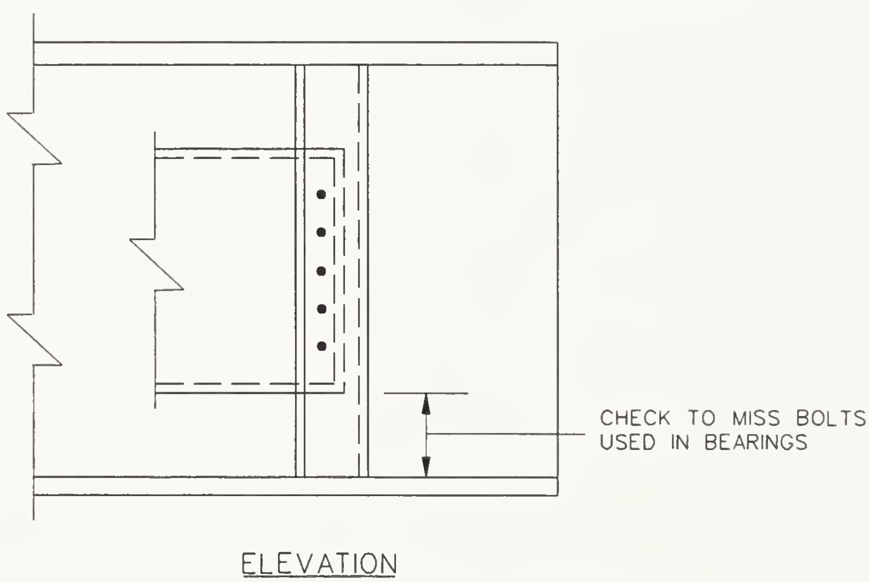
Beam	Diaphragm*	High-Strength Bolts
W920	C 460 x 63.5	5-M22
W840	C 460 x 63.5	5-M22
W760	C 380 x 50.4	4-M22
W690	C 380 x 50.4	4-M22
W610	C 310 x 30.8	3-M20

**Select a channel depth approximately one-half of the web depth.*

TYPICAL INTERMEDIATE DIAPHRAGM CONNECTION
(Rolled Beams)

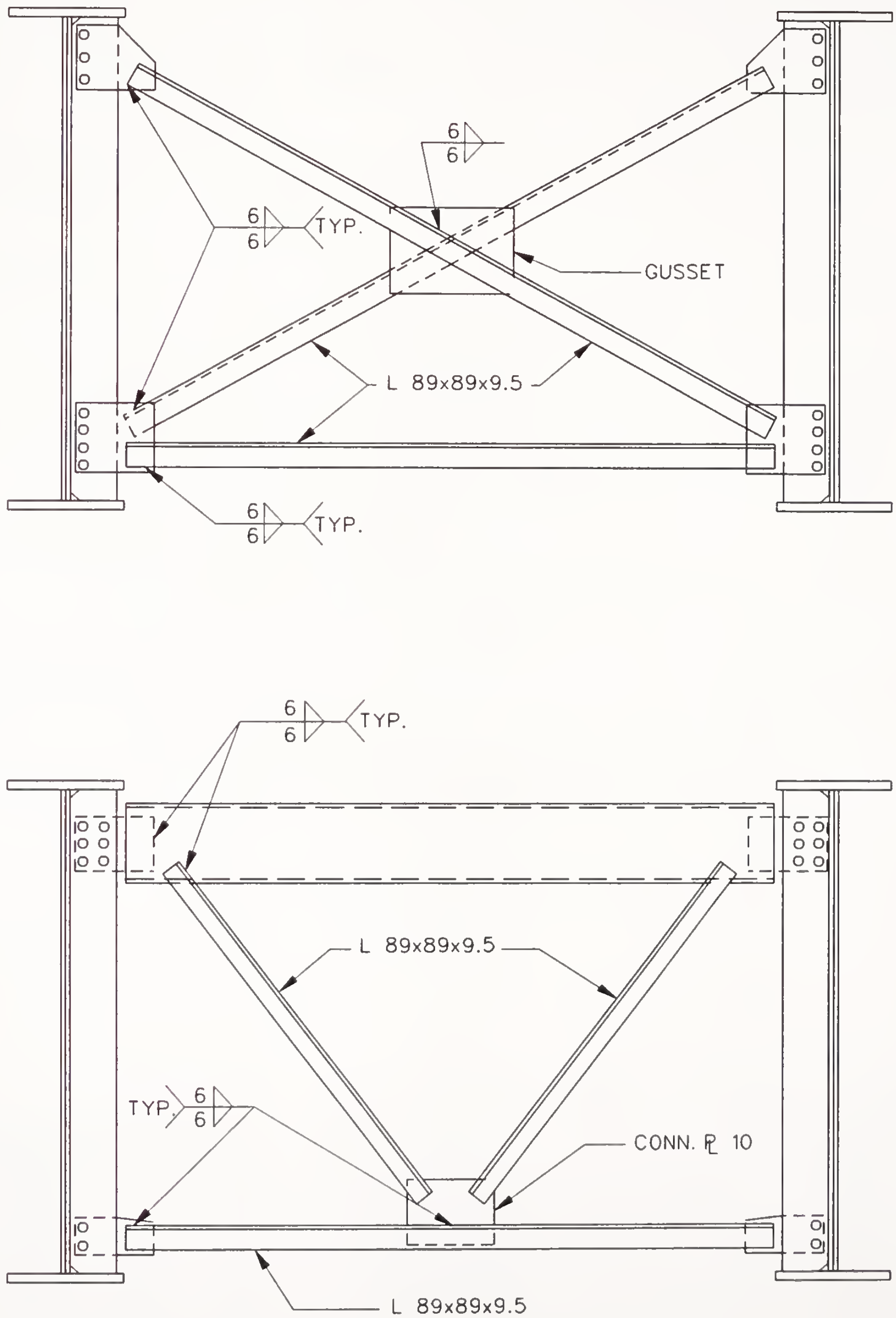
Figure 18.5D





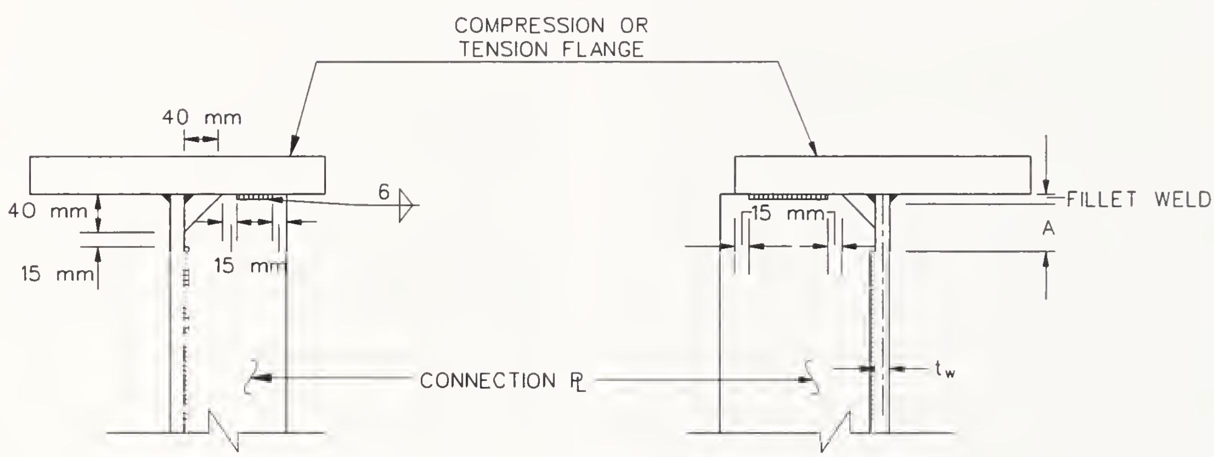
TYPICAL END DIAPHRAGM CONNECTION
(Rolled Beams)

Figure 18.5E



TYPICAL CROSS-FRAMES

Figure 18.5F



WELDED CONNECTION PLATE ATTACHMENT AT COMPRESSION OR TENSION FLANGE

Figure 18.5G

The width of connection plates should be sized to use bar stock and be not less than 125 mm. When the connection plate also acts as a transverse stiffener, it shall meet the requirements of LRFD Article 6.10.8.1.

18.5.5 Jacking

Reference: LRFD Article 3.4.3

The proper interpretation of the LRFD Specifications is that the plans should indicate designated points of jacking and whether or not the structure is capable of resisting 1.3 times the dead load reactions at those points. Slender beams may require web stiffeners at the jacking points. These stiffeners may either be part of the construction plans or fastened to the girder when and if the jacking is required. In general, jacking frames will not be required at the supports unless there is insufficient clearance between the bottom of beam and top of cap to place a jack. If less than 180 mm clearance is available for the jack, then the designer must decide whether the jack can be supported by temporary falsework. If temporary falsework is not feasible, then a jacking frame should be provided or the cap widened and the bearings placed on pedestals to provide sufficient space for a jack to be placed under the beam. Other locations where jacking may be required are:

1. at supports under expansion joints where joint leakage could deteriorate the girder bearing areas; and

2. at large displacement expansion bearings where deformation induced wear-and-tear is possible.

If no jacking frame is provided, then the cross-frame at the support still must be capable of transferring lateral wind forces to the bearings. For continuous structures using integral end bents, providing jacking frames at interior supports should not be considered.

18.5.6 Lateral Bracing

Reference: LRFD Article 6.7.5

The LRFD Specifications requires that the need for lateral bracing be investigated for all stages of assumed construction procedures and, if the bracing is included in the structural model used to determine force effects, it should be designed for all applicable limit states.

In general, lateral bracing is not required in the vast majority of steel girder bridges (short through medium spans). Typical diaphragms and cross-frames will transfer lateral loads adequately to eliminate the need for lateral bracing

Article 4.6.2.7 of the LRFD Specifications provides for various alternatives relative to lateral wind distribution in multi-girder bridges.

18.6 I-SECTIONS IN FLEXURE

Reference: LRFD Article 6.10

18.6.1 General

Reference: LRFD Article 6.10.1

18.6.1.1 Negative Flexural Deck Reinforcement

Reference: LRFD Article 6.10.3.7

Article 6.10.3.7 of the LRFD Specifications specifies that, in the negative moment area, the total cross sectional area of the longitudinal steel should not be less than 1% of the total cross sectional area of the deck slab (excluding the wearing surface) where the longitudinal tensile stress in the slab due to factored construction loads or the Service II load combination exceeds the factored modulus of rupture. However, the designer shall also ensure that sufficient negative moment steel is provided for the applied loads.

18.6.1.2 Rigidity in Negative Moment Areas

Reference: LRFD Articles 6.10.3.4 and 6.10.5

Article 6.10.3.4 of the LRFD Specifications permits assuming uncracked concrete in the negative moment areas for member stiffness. This is used to obtain continuity moments due to live load, future wearing surface and barrier weights placed on the composite section.

For the service limit state control of permanent deflections under LRFD Article 6.10.5 and the fatigue limit state under LRFD Article 6.6.1.2, the concrete slab may be considered fully effective for both positive and negative moments for members with shear connectors throughout their full lengths and satisfying LRFD Article 6.10.3.7.

18.6.2 Strength Limit States

Reference: LRFD Article 6.10.4

Moment redistribution according to elastic procedures (i.e., a 10% reduction in elastic negative support moments accompanied by a statically equivalent increase in the positive moments in adjacent spans) will be permitted for continuous spans if $F_y \leq 345$ MPa and if the negative moment support sections are compact.

18.6.3 Service Limit State Control of Permanent Deflection

Reference: LRFD Article 6.10.5

Moment redistribution is permitted for the investigation of permanent deflections.

18.6.4 Shear Connectors

Reference: LRFD Article 6.10.7.4

Shear connectors should be welded studs of which 22-mm diameter welded studs are preferred; the minimum diameter of studs is 19 mm. Shear connectors should have a minimum 64-mm concrete cover and should penetrate at least 50 mm above the bottom of the deck slab. The minimum longitudinal shear connector pitch is six stud diameters, and the maximum pitch is 600 mm.

The minimum number of studs in a group is two in a single transverse row. The transverse spacing, center to center, of the studs should be not less than four stud diameters. The minimum clear distance between the edge of the beam flange and the edge of the nearest stud shall be 25 mm. Details and spacing of stud shear connectors shall be detailed on the plans.

18.6.5 Stiffeners

Reference: LRFD Article 6.10.8

18.6.5.1 Transverse Intermediate Stiffeners

Reference: LRFD Article 6.10.8.1

Straight girders may be designed without intermediate transverse stiffeners, if economical, or with intermediate transverse stiffeners placed on one side of the web plate. In fact, if required, fascia girders should only have stiffeners on the inside face of the web. Due to the labor intensity of welding stiffeners to the web, the unit cost of stiffener by weight is approximately nine times that of the web. It is seldom economical to use the thinnest web plate permitted; therefore, the use of a thicker web and fewer intermediate transverse stiffeners, or no intermediate stiffeners at all, should be investigated. If it is decided to proceed with a design that requires stiffeners, the preferred width of the stiffener is one that can be cut from commercially produced bar stock.

Intermediate transverse stiffeners should be welded near side and far side to the compression flange. Transverse stiffeners need not and, for economical reasons, should not be welded to tension flanges. The distance between the end of the web-to-stiffener weld and the near toe of the web to flange fillet weld should be between $4t_W$ and $6t_W$. See Figure 18.6A for details.

Transverse stiffeners, except at diaphragm or cross-frame connections, should be placed on only one side of the web. The width of the projecting stiffener element, moment of inertia of the transverse stiffener and stiffener area shall satisfy the requirements of LRFD Article 6.10.8.1.

Longitudinal stiffeners used in conjunction with transverse stiffeners on spans over 80 m with deeper webs should preferably be placed on the opposite side of the web from the transverse stiffener. Where this is not practical (e.g., at intersections with cross-frame connection plates), the longitudinal stiffener should be continuous and not be interrupted for the transverse stiffener.

18.6.5.2 Bearing Stiffeners

Reference: LRFD Article 6.10.8.2

Bearing stiffeners are required at the bearing points of rolled beams and plate girders. Bearing stiffeners at integral end bents may be designed for dead load only. Design the stiffeners as columns and extend stiffeners to the outer edges of the bottom flange plates. The weld connecting the stiffener to the web should be designed to transmit the full bearing force from the stiffener to the web due to the factored loads. The bearing stiffeners may be either milled to fit against the flange through which they receive their reaction or welded to the flange with full penetration groove welds. See Figure 18.6B for details.

18.6.6 <u>Cover Plates</u>

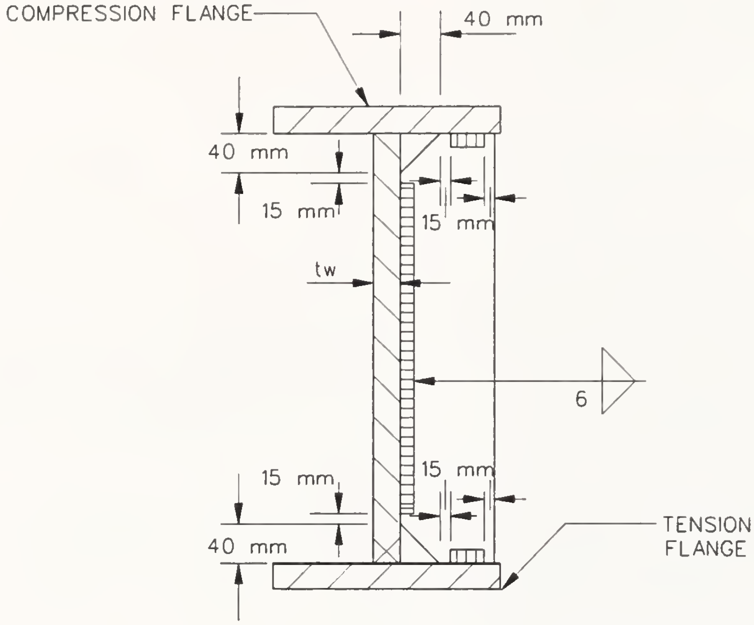
Reference: LRFD Article 6.10.9

Article 6.10.9.1 of the LRFD Specifications specifies that partial length cover plates should not be used with flange plates whose thickness exceeds 20 mm in non-redundant load path structures. According to LRFD Article 1.3.4, those elements and components whose failure is not expected to cause collapse of the bridge should not be designated as failure-critical and the associated structural system as redundant. The thickness of a single cover plate should not exceed twice the thickness of the flange plate. Multiple cover plates should not be employed. The width of the cover plate should be different from that of the flange plate to allow proper placement of the weld.

18.6.7 <u>Constructibility</u>

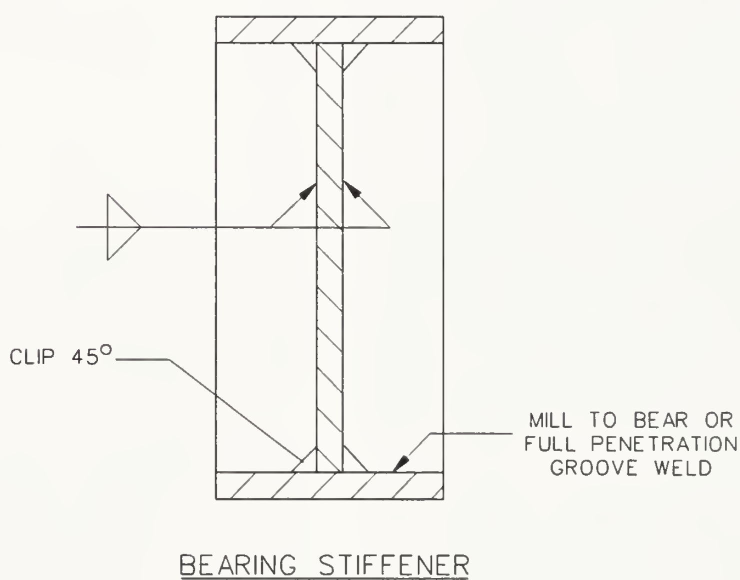
Reference: LRFD Article 6.10.3.2

Wind load, before the deck is placed, is transmitted to the piers by the structure acting as a lateral beam. Because of the diaphragms or



TRANSVERSE INTERMEDIATE STIFFENER DETAILS

Figure 18.6A



BEARING STIFFENER

Figure 18.6B

cross-frames present, the girders equally share the wind load. Normally, the structure can sustain this wind load without overstress.

18.6.8 Inelastic Analysis Procedures

Reference: LRFD Article 6.10.10

More thorough investigations using inelastic analysis procedures are generally not warranted and should not be used unless approved in advance by the Bridge Design Engineer.

18.7 CONNECTIONS AND SPLICES

Reference: LRFD Article 6.13

18.7.1 Bolted Connections

Reference: LRFD Article 6.13.2

The following applies to bolted connections:

1. Type. For unpainted weathering steel, A325 (Type 3) bolts should be used. For painted steel, A325 (Type 1) should be used.
2. Design. All bolted connections shall be designed as slip-critical at the Service II limit state, except for secondary bracing members.
3. Slip Resistance. LRFD Table 6.13.2.8-3 provides values for the surface condition (K_s). Use Class A surface condition for the design of slip-critical connections.

18.7.2 Welded Connections

Reference: LRFD Article 6.13.3

18.7.2.1 Welding Process

Welding is performed by fusing two pieces of metal together so that they become continuous. The process of melting and cooling resembles the conditions under which the steel was originally made, and the characteristics of the weld can be very much like the adjacent steel.

Welded connections are the most common types of connections used in shop fabrication today. During the 1950s, welding fabrication replaced riveted fabrication as steel specifically formulated for weldability replaced A7 and other older steel chemistries.

The governing specification for welding is the ANSI/AASHTO/AWS **Bridge Welding Code D1.5**. This single specification contains almost all requirements for the welding fabrication of bridges. The engineer needs to be aware, however, that this specification does not provide control over all of the welding issues that may arise on a project. Additional reference specifications that may need to be consulted are:

1. AWS D1.1 for welding of tubular members and strengthening or repair of existing structures, and
2. AWS D1.4 if a situation arises where the welding of reinforcing steel must be covered by a specification.

This Code accepts as *prequalified* (i.e., acceptable without further proof of suitability if applied under specified conditions) four welding processes using electric arcs. These are:

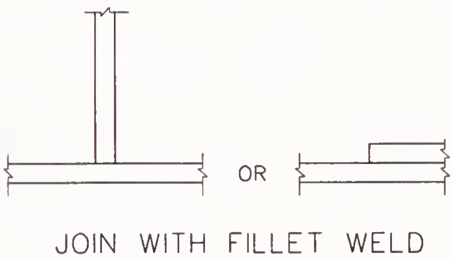
1. shielded metal arc welding (SMAW). This process is also known as stick welding and is what is commonly considered welding;
2. submerged arc welding (SAW);
3. gas metal arc welding (GMAW). This process is also sometimes called *metal inert gas welding* or MIG; and
4. flux-cored arc welding (FCAW).

Of these, SMAW is the principal method for hand welding; the others are automatic or semi-automatic processes. Shop practice on most weldments is automatic, offering the advantages of much higher speed and greater reliability. Hand welding is mostly limited to short production welds or tack welds during fitting up components prior to production welding.

Acceptable procedures for using these processes or others requires testing of the welding operations and of welds, using a filler metal that is compatible with the base metal, proper preparation of the joints, controlling the temperature and rate of welding, and control of the welding process.

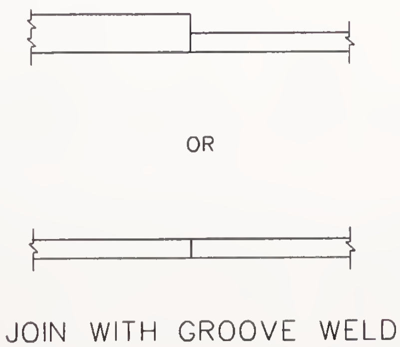
18.7.2.2 Welds for Bridges

The primary types of welds used in bridge fabrication are fillet welds and butt (or groove) welds. One of these two types accounts for approximately 80% of all bridge welding. A typical cross section where specification of a fillet weld is appropriate is shown in Figure 18.7A, and a typical cross section where specification of a butt or groove weld is appropriate is shown in Figure 18.7B.



FILLET WELD

Figure 18.7A



BUTT WELD

Figure 18.7B

18.7.2.3 Welding Symbols

Welding symbols are used as an instruction on the form, size and other characteristics of the desired weld. The forms of the symbols are precisely defined by AWS A2.4. When these symbols are properly used, the meaning is clear and unambiguous. If not used exactly as prescribed, the meaning may be ambiguous, leading to problems for all involved. The **AISC Manual** and most steel design textbooks have examples of welding symbols which, although technically correct, are more complicated than the typical engineer needs. With minor modifications, the examples in Figure 18.7C will suffice for approximately 80% of bridge fabrication circumstances.

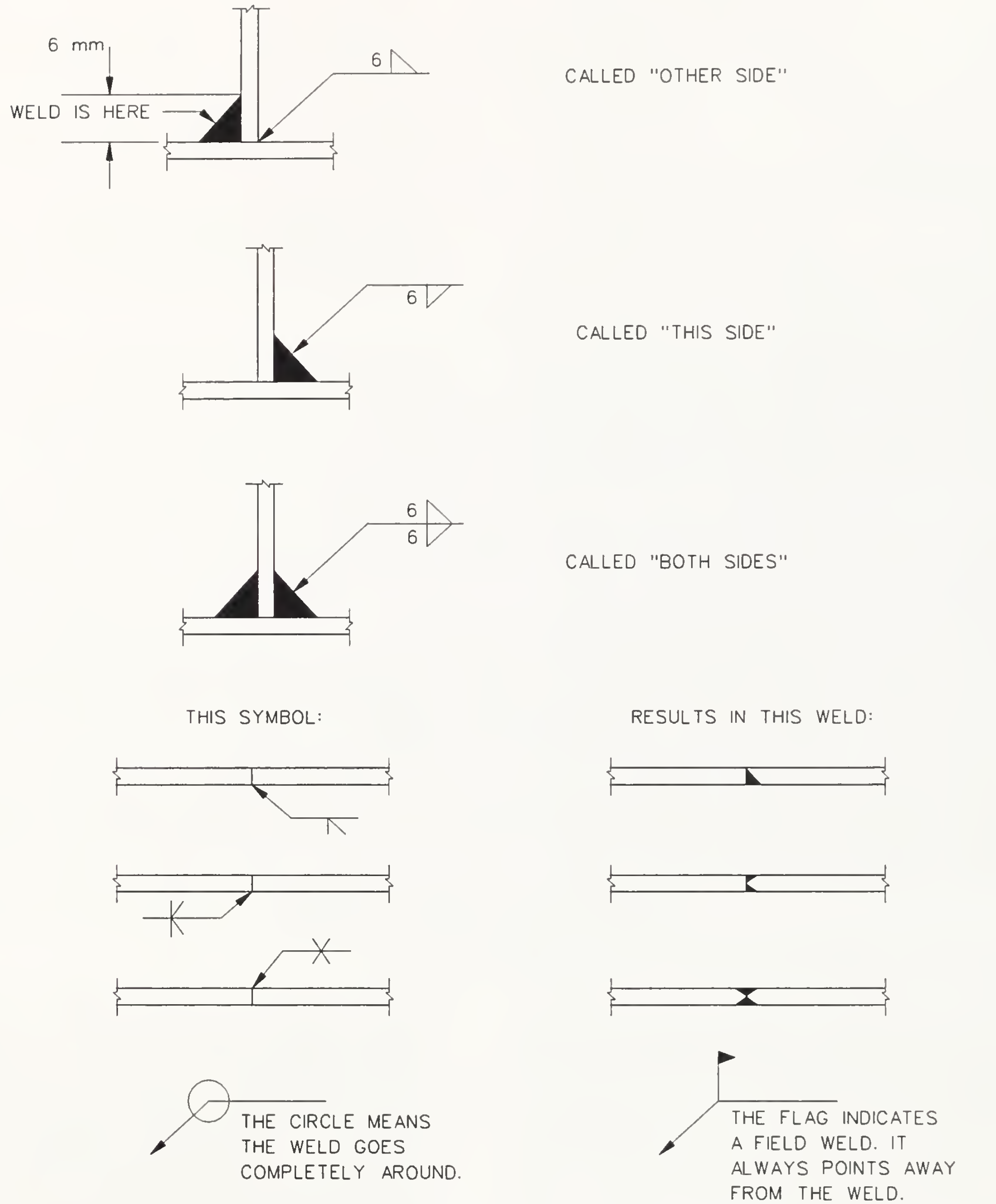
18.7.2.4 Electrode Nomenclature

The strength of the weld filler metal is known from the electrode designation. Figure 18.7D illustrates the standard nomenclature to identify electrodes. The figure represents more than a designer typically needs to know but, as an illustration, the standard MDT pile weld note says use E7018 or E7028 series electrodes. This means that electrodes with a weld-metal strength of 70,000 psi and the indicated welding procedures for all positions of welding or only flat and horizontal positions, respectively.

18.7.2.5 Design of Welds

The design of the weld is integral to the LRFD Section on Steel Design. The AASHTO Specifications addresses topics such as resistance factors for welds, minimum weld size and weld details to reduce fatigue susceptibility.

To proceed with the calculations required by the specification, the concept is that the strength of a welded connection is dependent on the weld metal strength and the area of the weld that



"THIS SIDE" AND "OTHER SIDE" WELDS ARE THE SAME SIZE UNLESS SPECIFIED OTHERWISE.

SYMBOLS APPLY BETWEEN ABRUPT CHANGES IN DIRECTION OF WELDING UNLESS GOVERNED BY THE "ALL-AROUND SYMBOL" OR OTHERWISE DIMENSIONED.

WELDING SYMBOLS

Figure 18.7C

These digits indicates the following:	
Exx1z	All positions of welding
Exx2z	Flat and horizontal positions
Exx3z	Flat welding positions only
These digits indicate the following:	
Exx10	DC, reverse polarity
Exx11	AC or DC, reverse polarity
Exx12	DC straight polarity, or AC
Exx13	AC or DC, straight polarity
Exx14	DC, either polarity or AC, iron powder
Exx15	DC, reverse polarity, low hydrogen
Exx16	AC or DC, reverse polarity, low hydrogen
Exx18	AC or DC, reverse polarity, iron powder, low hydrogen
Exx20	DC, either polarity, or AC for horizontal fillet welds; and DC either polarity, or AC for flat position welding
Exx24	DC, either polarity, or AC, iron powder
Exx27	DC, straight polarity, or AC for horizontal fillet welding; and DC, either polarity, or AC for flat position welding, iron powder
Exx28	AC or DC, reverse polarity, iron powder, low hydrogen

The "xx" shown above is a two-digit number indicating the weld metal tensile strength in 1000 psi (6.9 MPa) increments. For example, E7018 is 70,000 psi (483 MPa).

ELECTRODE NOMENCLATURE

Figure 18.7D

resists the load. Weld metal strength is a fairly self-defining term. The area of the weld that resists load is a product of the theoretical throat multiplied by the length. The theoretical weld throat is the minimum distance from the root of the weld to its theoretical face. Fillet welds resist load through shear on the throat, while groove welds resist load through tension, compression or shear depending upon the application.

Often, it is best only to show the type and size of weld required and leave the details to the fabricator.

When considering design options, note that the most significant driving factor in the cost of a weld is the volume of the weld material that is deposited. Over specifying a welded joint is unnecessary and uneconomical. Welds sized to be made in a single pass are preferred. The weld should be designed economically, but its size should not be less than 6 mm and, in no case, less than the requirements of LRFD Article 6.13.3.4 for the thicker of the two parts joined. Weld terminations should be shown.

The following prohibitions should be observed:

1. Field Welding. Field welding is prohibited for all splices.

2. Intersecting Welds. These should be avoided, if practical.

3. Intermittent Fillet Welds. These are prohibited.

4. Partial Penetration Groove Welds. These are prohibited except as permitted for orthotropic steel decks under LRFD Article 9.8.3.7.2.

Provide careful attention to the accessibility of welded joints. Provide sufficient clearance to enable a welding rod to be placed at the joint. Often, a large-scale sketch or an isometric drawing of the joint will reveal difficulties in welding or where critical weld stresses must be investigated.

18.7.2.6 Inspection and Testing

Indispensable to the reliable use of welding is a systematic program of inspection and testing. Inspection is done at the shop and at the field site. The function of the inspection is to guarantee that specified materials and procedures are used under conditions where proper welding is possible. If the sequence of welding has been specified, the inspector should be able to certify conformance.

In spite of careful inspection, weld defects may escape detection unless all or part of the work is subjected to tests. There are two broad categories of testing — destructive testing, which is used very sparingly for big problems or forensic studies, and nondestructive testing, which is used extensively to guarantee the quality of the welds. The Department routinely uses the following types of non-destructive testing (NDT):

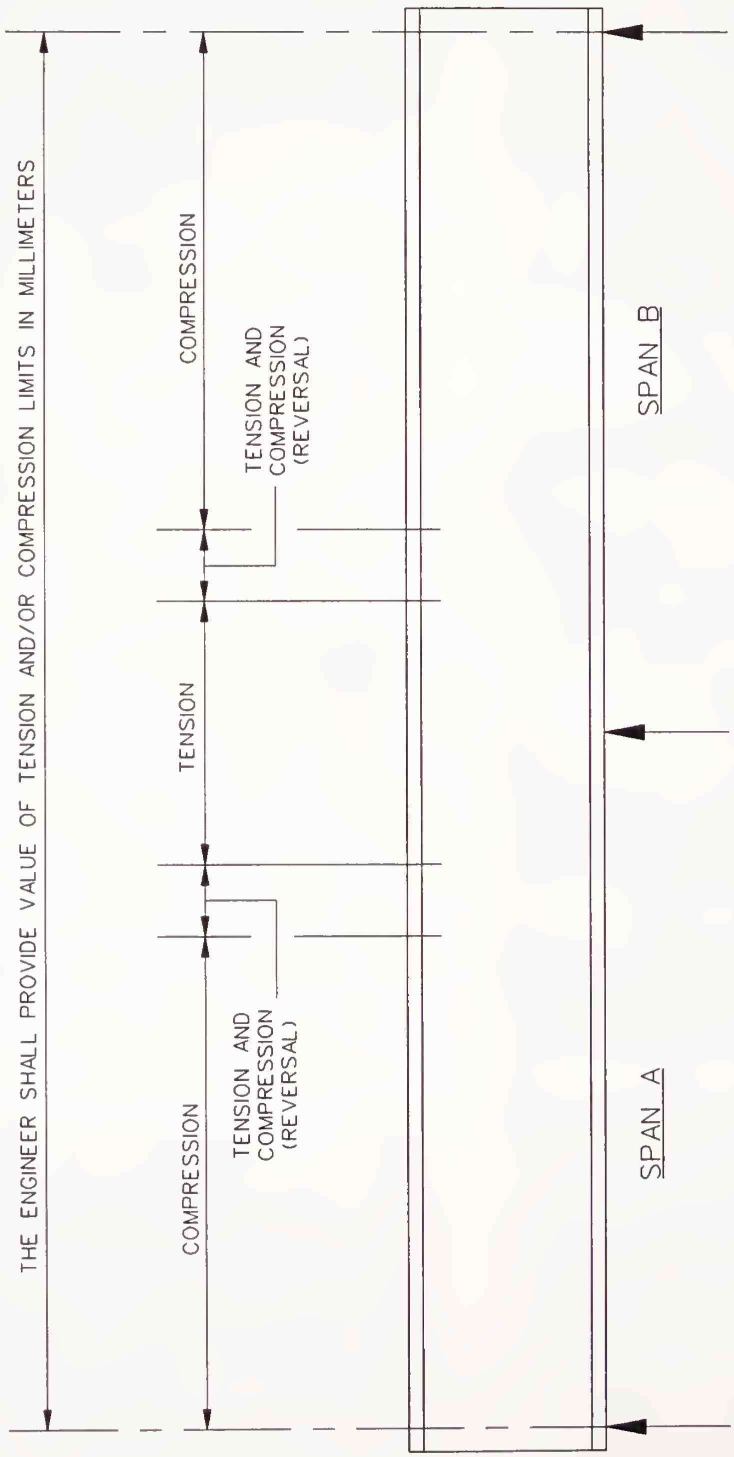
1. Radiography (RT). Used to find cracks and inclusions after a weld is completed. The process involves placing film on one side of the weld and a source of gamma or x-rays on the other side of the weld. Shadows on the exposed film indicate cracks or inclusions in the welds or adjacent areas. RT is most effective on full penetration butt joints with ready access to both sides.

2. Ultrasonic Testing (UT). Relies on the reflection patterns of high-frequency sound waves, which are transmitted at an angle through the work. Cracks and defects interrupt the sound transmission, altering the display on an oscilloscope. This method can reveal many defects that the other methods do not, but it relies very heavily on the interpretative skill of the operator.

3. Magnetic Particle (MT). Done by covering the surface of the weld with a suspension of ferromagnetic particles and then applying a strong magnetic field. Cracks in the weld interrupt the magnetic force lines, causing the particles to concentrate in the vicinity in patterns easily interpreted by the inspector.

4. Dye Penetrant (DP). Uses a dye in liquid form to detect cracks. Capillary tension in the liquid causes the dye to penetrate into the crack, remaining behind after the surface is cleaned.

To aid the inspector, the plans for continuous structures should include a sketch showing the location of compression, reversal and tension regions along the girder top flange. Show the length of each stress region and reference them to the point of support. Figure 18.7E illustrates the information required.



SCHEMATIC OF TOP FLANGE STRESS

Figure 18.7E

18.7.3 Splices

Reference: LRFD Article 6.13.6

Significant revisions have been made to the provisions for the design of bolted field splices (LRFD Article 6.13.6.1) in the 1999 Interims to the LRFD Specifications. The American Iron and Steel Institute (AISI) has developed software called AISI splice incorporating the revisions. The program is available from AISI for the cost of handling. These revisions were made to:

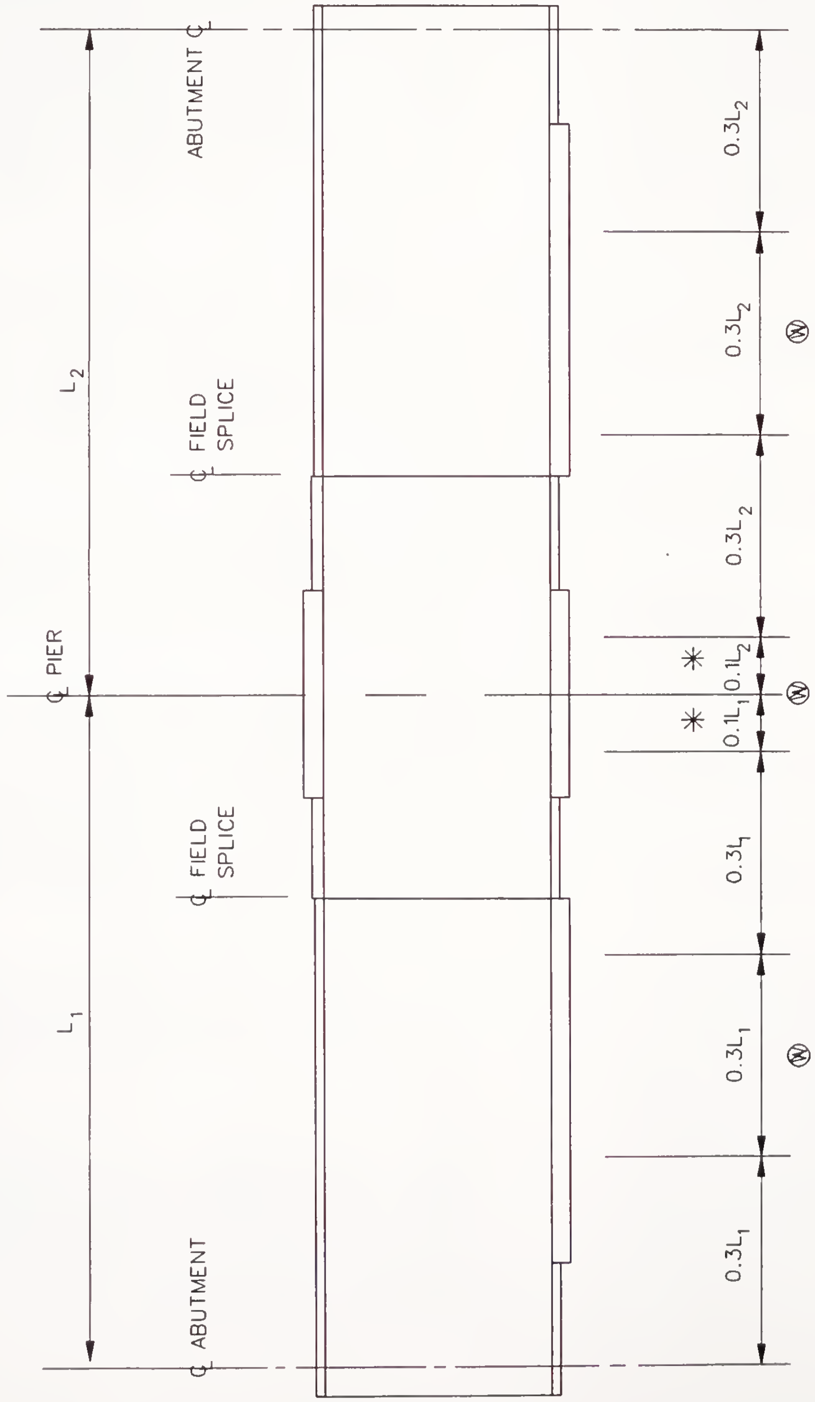
1. ensure a more consistent interpretation for the design of splices in flexural members at all limit states;
2. better handle the design of splices for composite flexural members, especially in areas of stress reversal;
3. provide a more consistent and reasonable design shear for splices in flexural members; and
4. better determine the effective flange area to be used for flexural members with holes.

In addition to the provisions of LRFD Article 6.13.6, the following will apply to splices:

1. Location. In general, field splices should be located at low-stress areas and near the points of dead-load contraflexure for continuous spans. Numerous butt welds and/or butt welds located in high stress regions are not desirable. The location of shop butt splices is normally dependent upon the length of plate available to the fabricator. This length varies depending upon the rolling process. The maximum length of normalized and quenched and tempered plates is 15 m. Other plates can be obtained in lengths greater than 25 m depending on thickness. The cost of adding a shop welded splice instead of extending a thicker plate should be considered when designing members. Discussion with a fabricator or the NSBA during the design is suggested.

   To assist the fabricator and contractor, the designer should provide an indication of where splices are acceptable and not acceptable. Figure 18.7F presents a typical detail that may be included in the contract plans. In addition, the plans should note that a plate as long as practical should be used, eliminating as many shop splices as possible.

2. Swept Width. The swept width is equal to the sweep in a curved girder plus the flange width. On curved girders, the swept width between splices should generally be limited to 3 m to accommodate the shipment of the steel.
3. Bolts. Bolt loads shall be calculated by an elastic method of analysis. Provide no less than two lines of bolts on each side of the web splice.
4. Composite Girder. If a composite girder is spliced at a section where the moment can be resisted without composite action, the splice may be designed as noncomposite. If composite action is necessary to resist the loads, the splice should be designed for the forces due to composite action.
5. Design. Bolted splices must be slip-critical under Service II loads and must be designed as a bearing type connection under strength limit states.
6. Welded Shop Splice. Figure 18.7G illustrates welded splice details. See LRFD Article 6.13.6.2 for more information regarding splicing different thicknesses of material using butt welds.

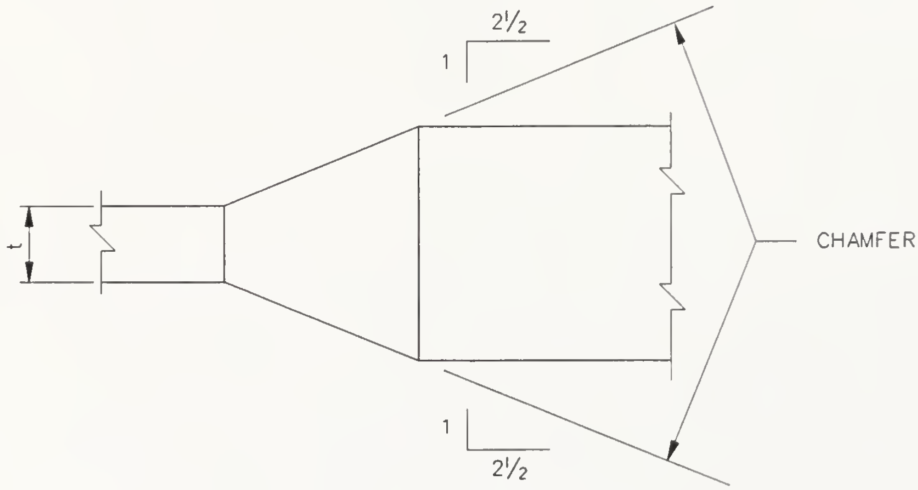


Ⓦ DENOTES REGIONS WHERE WELDED SHOP SPLICES ARE NOT ALLOWED IN FLANGES OR WEBS. USE AS LONG AS PRACTICAL. KEEP THE NUMBER OF SHOP SPLICES TO A MINIMUM.

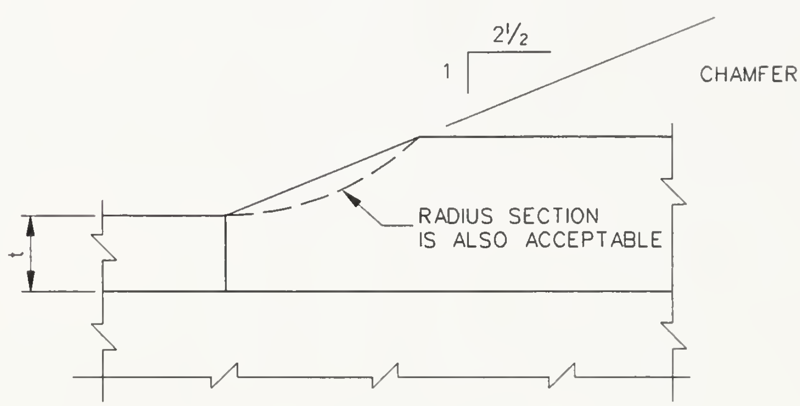
* NOTE: THE LIMITS SHOWN ARE DEPENDENT ON SPAN LENGTH, WHETHER IT IS POSITIVE OR NEGATIVE MOMENT, X-FRAME LOCATION, SECTION CHANGES AND GENERAL GOOD JUDGEMENT.

LOCATION OF SPLICES

Figure 18.7F



WEB SPLICE DETAILS



FLANGE SPLICE DETAILS

TYPICAL SUBMERGED ARC WELD DETAILS

Figure 18.7G